FIRE RESISTANCE ASSESSMENT OF PARTIALLY PROTECTED COMPOSITE FLOORS (FRACOF) DESIGN GUIDE

FOREWORD

FIRE RESISTANCE ASSESSMENT OF PARTIALLY PROTECTED COMPOSITE FLOORS (FRACOF)

Contents

SUMMARY

Large-scale fire tests conducted in a number of countries and observations of actual building fires have shown that the fire performance of composite steel framed buildings is much better than is indicated by fire resistance tests on isolated elements. It is clear that there are large reserves of fire resistance in modern steel-framed buildings and that standard fire resistance tests on single unrestrained members do not provide a satisfactory indicator of the performance of such structures.

This publication presents guidance on the application of a simple design method, as implemented in FRACOF software, that has been developed as a result of observation and analysis of the BRE Cardington large-scale building fire test programme carried out during 1995 and 1996. The recommendations are conservative and are limited to structures similar to that tested, i.e. non-sway steel-framed buildings with composite floors. The guidance gives designers access to whole building behaviour and allows them to determine which members can remain unprotected while maintaining levels of safety equivalent to traditional methods.

In recognition that many fire safety engineers are now considering natural fires, a natural fire model is included alongside the use of the standard fire model, both expressed as temperature-time curves in Eurocode 1.

In addition to the design guidance provided by this publication, a separate Engineering Background document provides details of fire testing and finite element analysis conducted as part of the FRACOF project and some details of the Cardington tests which were conducted on the eight-storey building at Cardington. The background document will assist the reader to understand the basis of the design recommendations in this publication.

1 INTRODUCTION

The design recommendations in this publication are based on the performance of composite floor plates, as interpreted from actual building fires and from full-scale fire tests[1,2,3]. These conservative recommendations for fire design may be considered as equivalent to advanced methods in the Eurocodes.

The elements of structure of multi-storey buildings are required by national building regulations to have fire resistance. The fire resistance may be established from performance in standard fire resistance tests or by calculations in accordance with recognised standards, notably EN1991-1-2[4], EN 1993-1-2[5] and EN 1994-1-2[6]. In a standard fire test, single, isolated and unprotected I or H section steel beams can only be expected to achieve 15 to 20 minutes fire resistance. It has thus been normal practice to protect steel beams and columns by use of fire resisting boards, sprays or intumescent coatings, or, in slim floor or shelf angle floor construction, by encasing the structural elements within floors.

Large-scale natural fire tests[7] carried out in a number of countries have shown consistently that the inherent fire performance of composite floor plates with unprotected steel elements is much better than the results of standard tests with isolated elements would suggest. Evidence from real fires indicates that the amount of protection being applied to steel elements may be excessive in some cases. In particular, the Cardington fire tests presented an opportunity to examine the behaviour of a real structure in fire and to assess the fire resistance of unprotected composite structures under realistic conditions.

As the design recommendations given in this publication are related to generalised compartment fire, they can be easily applied under standard fire condition such as it is demonstrated through the real scale floor test within the scope of FRACOF project. Obviously, this possibility provides a huge advantage to engineers in their fire safety design of multi-storey buildings with steel structures.

Where national building regulations permit performance-based design of buildings in fire, the design method provided by this guide may be applied to demonstrate the fire resistance of the structure without applied fire protection. In some countries acceptance of such demonstration may require special permission from the national building control authority.

The recommendations presented in this publication can be seen as extending the fire engineering approach in the area of structural performance and developing the concept of fire safe design. It is intended that designs carried out in accordance with these recommendations will achieve at least the level of safety required by national regulations while allowing some economies in construction costs.

In addition to fire resistance for the standard temperature-time curve, recommendations are presented for buildings designed to withstand a natural fire. Natural fires can be defined in the FRACOF software using the parametric temperature-time curve given in EN1991-1-2. This takes account of the size of the compartment, the size of any openings and the amount of combustibles. Alternatively, the FRACOF software permits temperature-time curves to be read from a text file, allowing output from other fire models to be used.

The recommendations apply to composite frames broadly similar to the eight-storey building tested at Cardington, as illustrated in Figure 1.1 and Figure 1.2.

The design recommendations are presented as guide to the application of the FRACOF software, which is available as a free download from *www.arcelormittal.com/sections*.

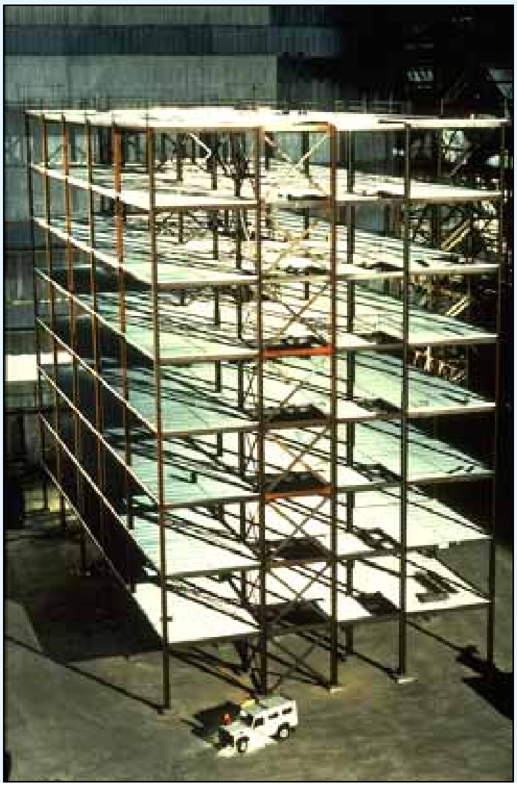


Figure 1.1 Cardington test building prior to the concreting of the floors

Figure 1.2 View of unprotected steel structure

1.1 UK national regulations

The Building Regulations in England and Wales changed from prescriptive to performance based requirements in 1991. The statutory requirements state that "The building shall be designed and constructed so that, in the event of fire, its stability will be maintained for a reasonable period." Approved Document B[(8)] gives practical guidance with respect to the statutory requirements and states that "A fire safety engineering approach that takes into account the total fire safety package can provide an alternative approach to fire safety."

The regulations in Scotland and the regulations in Northern Ireland have recently changed and are now, like Approved Document B, based on the test of "reasonableness" and allow a fire safety engineering approach to be used.

1.2 French national regulations

The regulations in France for fire resistance introduced performance based requirements in 2004 in addition to prescriptive requirements. The statutory requirement states that, *The building structure shall be designed and constructed so that, in the event of fire, its stability will be maintained for the whole period of fire if a real scenario is applied.* The Ministerial Order of 21 March 2004 gives practical guidance with respect to the statutory requirements and states that a fire safety engineering approach for fire resistance that takes into account the natural fire can provide an alternative approach to fire resistance safety, provided that

- the fire scenario is approved by the fire safety commission;
- the fire safety engineering study is checked by an approved laboratory;
- special terms and conditions for future exploitation of investigated buildings are specified in an individual document.

2 BASIS OF DESIGN

This Section gives an overview of the design principles and assumptions underlying the development of the simple design method; more detailed information is given in the accompanying background document [(7)]. The type of structure that the design guidance is applicable to is also outlined.

The design guidance has been developed from research based on the results from fire tests, ambient temperature tests and finite element analyses.

2.1 Fire safety

The design recommendations given in the simple design method have been prepared such that the following fundamental fire safety requirements are fulfilled:

- There should be no increased risk to life safety of occupants, fire fighters and others in the vicinity of the building, relative to current practice.
- On the floor exposed to fire, excessive deformation should not cause failure of compartmentation, in other words,. the fire will be contained within its compartment of origin and should not spread horizontally or vertically.

2.2 Type of structure

The design guidance given in the simple design method applies only to steel-framed buildings with composite floor beams and slabs of the following general form:

- braced frames not sensitive to buckling in a sway mode,
- frames with connections designed using simple joint models,
- composite floor slabs comprising steel decking, a single layer of reinforcing mesh and normal or lightweight concrete, designed in accordance with EN1994-1-1[(9)],
- floor beams designed to act compositely with the floor slab and designed to EN 1994-1-1.

The guidance does **not** apply to:

- floors constructed using precast concrete slabs,
- internal floor beams that have been designed to act non-compositely (beams at the edge of the floor slab may be non-composite),
- beams with service openings.

2.2.1 Simple joint models

The joint models adopted during the development of the guidance given in this publication assume that bending moments are not transferred through the joint. The joints are known as 'simple'.

Beam-to-column joints that may be considered as 'simple' include joints with the following components:

- Flexible end plates (Figure 2.1)
- Fin plates (Figure 2.2)
- Web cleats (Figure 2.3)

Further information on the design of the components of 'simple' joints is given in Section 3.6.

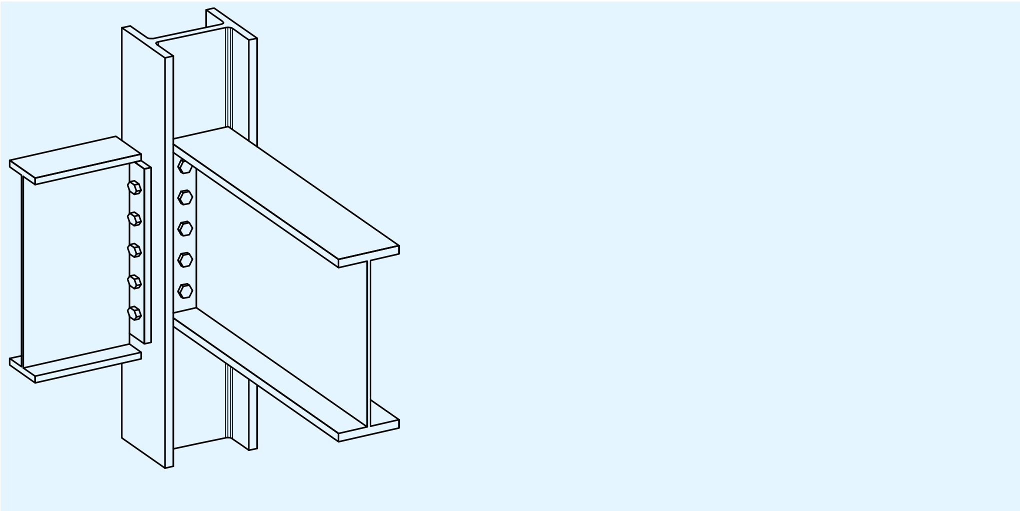


Figure 2.1 Example of a joint with flexible end plate connections

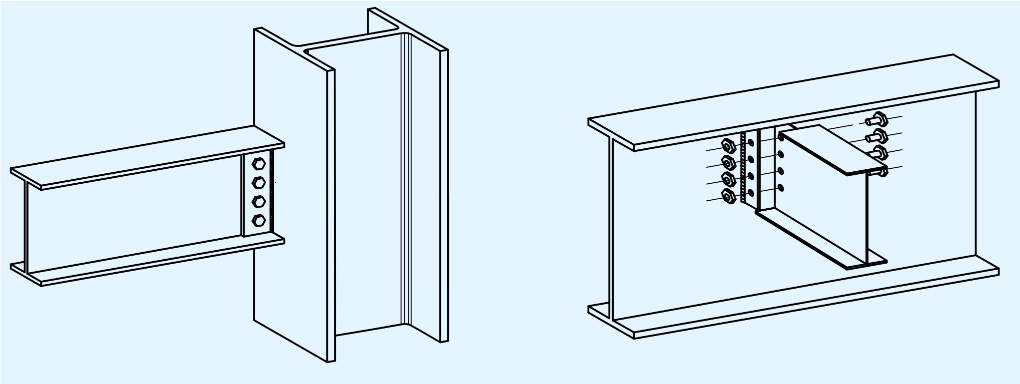


Figure 2.2 Examples of joints with fin plate connections

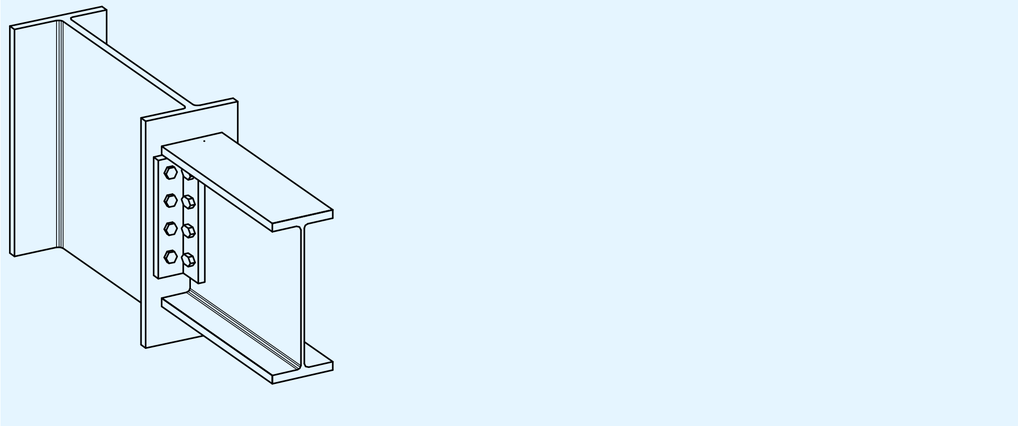


Figure 2.3 Example of a joint with a web cleat connection

2.2.2 Floor slabs and beams

The design recommendations given in this guide are applicable to profiled steel decking up to 80 mm deep with depths of concrete above the steel decking from 60 to 90 mm. The resistance of the steel decking is ignored in the fire design method but the presence of the steel decking prevents spalling of the concrete on the underside of the floor slab. This type of floor construction is illustrated in Figure 2.4.

The design method can be used with either isotropic or orthotropic reinforcing mesh, that is, meshes with either the same or different areas in orthogonal directions. The steel grade for the mesh reinforcement should be specified in accordance with EN10080. As the design method requires ductile mesh reinforcement in order to accommodate large slab deflections Class B or Class C should be specified. The FRACOF software can only be used for welded mesh reinforcement and can not consider more than one layer of reinforcement. Reinforcement bars in the ribs of the composite slab are **not** required**.**

The software includes A and B series standard fabric meshes as defined by UK national standards[11,12] (Table 2.1) and a range of mesh sizes defined by French national standards[13,14] (Table 2.2), and commonly used in the French construction market. User defined sizes of welded mesh are also permitted in the FRACOF software.

Table 2.1 Fabric mesh as defined by BS 4483[11]

Mesh Reference	Size of mesh (mm)	Weight (kg/m^2)	Longitudinal wires		Transverse wires	
			Size (mm)	Area (mm^2/m)	Size (mm)	Area (mm^2/m)
A142	200x200	2.22	6	142	6	142
A193	200x200	3.02	7	193	7	193
A252	200x200	3.95	8	252	8	252
A393	200x200	6.16	10	393	10	393
B196	100x200	3.05	5	196	7	193
B283	100x200	3.73	6	283	7	193
B385	100x200	4.53	7	385	7	193
B503	100x200	5.93	8	503	8	252

Table 2.2 Fabric mesh commonly used in French market

Mesh Reference	Size of mesh (mm)	Weight (kg/m^2)	Longitudinal wires		Transverse wires	
			Size (mm)	Area (mm^2/m)	Size (mm)	Area (mm^2/m)
ST 20	150x300	2.487	6	189	7	128
ST 25	150x300	3.020	7	257	7	128
ST 30	100x300	3.226	6	283	7	128
ST 35	100x300	6.16	7	385	7	128
ST 50	100x300	3.05	8	503	8	168
ST 60	100x300	3.73	9	636	9	254
ST 15 C	200x200	2.22	6	142	6	142
ST 25 C	150x150	4.03	7	257	7	257
ST 40 C	100x100	6.04	7	385	7	385
ST 50 C	100x100	7.90	8	503	8	503
ST 60 C	100x100	9.98	9	636	9	636

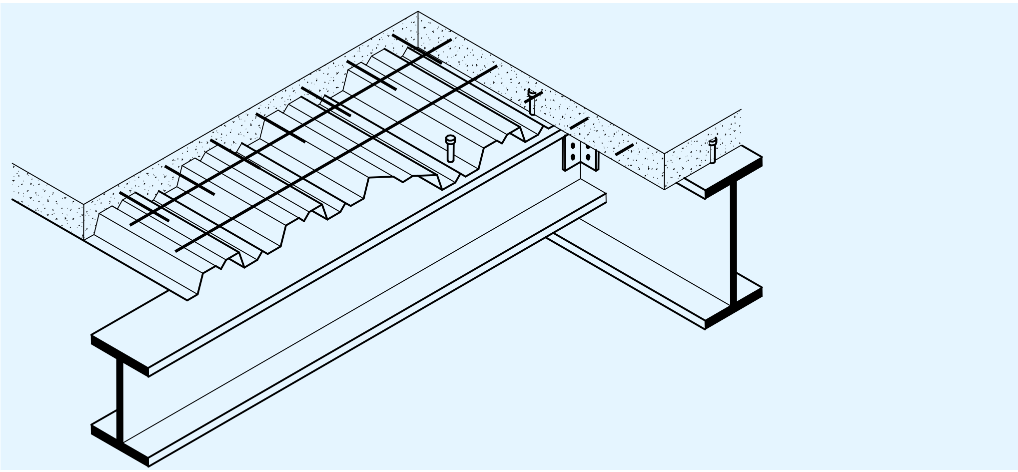


Figure 2.4 Cut away view of a typical composite floor construction

It is important to define the beam sizes used in the construction of the floor plate as this will influence the fire performance of the floor plate. The designer will need to have details of the serial size, steel grade and degree of shear connection available for each beam in the floor plate. The FRACOF software interface allows the user to choose from a predefined list of serial sizes covering common British, European and American I and H sections.

2.3 Floor design zones

The design method requires the designer to split the floor plate into a number of floor design zones as shown in Figure 2.5. The beams on the perimeter of these floor design zones must be designed to achieve the fire resistance required for the floor plate and will therefore normally be fire protected.

A floor design zone should meet the following criteria:

- Each zone should be rectangular.
- Each zone should be bounded on all sides by beams.
- The beams within a zone should only span in one direction.
- Columns should not be located within a floor design zone; they may be located on the perimeter of the floor design zone.
- For fire resistance periods in excess of 60 minutes, or when using the parametric temperature-time curve, all columns should be restrained by at least one fire protected beam in each orthogonal direction.

All internal beams within the zone may be left unprotected, provided that the fire resistance of the floor design zone is shown to be adequate using the FRACOF software. The size and spacing of these unprotected beams are not critical to the structural performance in fire conditions.

An example of a single floor design zone is given in Figure 2.5.

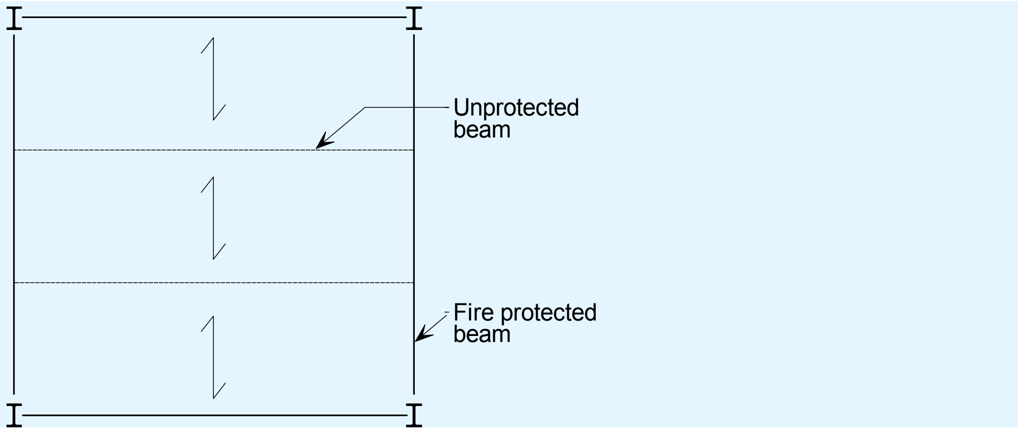


Figure 2.5 Example of a floor design zone

2.4 Combination of actions

The combination of actions for accidental design situations given in 6.4.3.3 and Table A1.3 of EN 1990 (15) should be used for fire limit state verifications. With only unfavourable permanent actions and no prestressing actions present, the combination of actions to consider is:

$$\sum G_{\mathrm{k},j,\mathrm{sup}} + A_{\mathrm{d}} + (\psi_{1,1} \text{ or } \psi_{2,1})Q_{\mathrm{k},1} + \sum \psi_{2,i} Q_{\mathrm{k},i}$$

Where:

$G_{\mathrm{k},j,\mathrm{sup}}$	Unfavourable permanent action
A_{d}	Leading accidental action
$Q_{\mathrm{k},1}$ and $Q_{\mathrm{k},i}$	Accompanying variable actions, main and other respectively

$\psi_{1,1}$	Factor for the frequent value of the leading variable action
$\psi_{2,i}$	Factor for the quasi-permanent value of the *i*th variable action

The use of either $\psi_{1,1}$ or $\psi_{2,1}$ with $Q_{k,1}$ should be specified in the relevant National Annex. The National Annex for the country where the building is to be constructed should be consulted to determine which factor to use.

The values used for the ψ factors relate to the category of the variable action they are applied to. The Eurocode recommended values for the ψ factors for buildings are given in Table A1.1 of EN 1990; those values are confirmed or modified by the relevant National Annex. The ψ factor values for buildings in the UK and France are summarised in Table 2.3. For floors that allow loads to be laterally distributed, the following uniformly distributed loads are given for moveable partitions in 6.3.1.2(8) of EN 1991-1-1[(16)]:

Movable partitions with a self-weight ≤ 1,0 kN/m wall length: $q_k = 0{,}5$ kN/m^2

Movable partitions with a self-weight ≤ 2,0 kN/m wall length: $q_k = 0{,}8$ kN/m^2

Movable partitions with a self-weight ≤ 3,0 kN/m wall length: $q_k = 1{,}2$ kN/m^2.

Moveable partitions with self-weights greater than 3.0 kN/m length should be allowed for by considering their location.

The Eurocode recommended values for variable imposed loads on floors are given in Table 6.2 of EN 1991-1-1; those values may also be modified by the relevant National Annex. Table 2.4 presents the Eurocode recommended values and the values given in the UK and French National Annexes for the imposed load on an office floor.

Table 2.3 Values of ψ factors

Actions	Eurocode recommended values		UK National Annex values		French National Annex values	
	ψ_1	ψ_2	ψ_1	ψ_2	ψ_1	ψ_2
Domestic, office and traffic areas where: 30 kN < vehicle weight ≤ 160 kN	0.5	0.3	0.5	0.3	0.5	0.3
Storage areas	0.9	0.8	0.9	0.8	0.9	0.8
Other*	0.7	0.6	0.7	0.6	0.7	0.6

* Climatic actions are not included

Table 2.4 Imposed load on an office floor

Category of loaded area	Eurocode recommended values		UK National Annex values		French National Annex values	
	q_k (kN/m^2)	Q_k (kN)	q_k (kN/m^2)	Q_k (kN)	q_k (kN/m^2)	Q_k (kN)
B – Office areas	3.0	4.5	2.5* or 3.0**	2.7	3.5 – 5.0	15.0

* Above ground floor level

**At or below ground floor level

2.5 Fire exposure

The recommendations given in the simple design method may be applied to buildings in which the structural elements are considered to be exposed to a standard temperature-time curve or parametric temperature-time curve, both as defined in EN 1991-1-2. Advanced model may also be used to define a temperature –time curve for a natural fire scenario. The resulting temperature-time time curve may be input to the FRACOF software in the form of a text file.

In all cases, the normal provisions of national regulations regarding means of escape should be followed.

2.5.1 Fire resistance

The Cardington fire tests were conducted using both real ('natural') fires and non standard gas fires. The tests did not follow the standard temperature-time curve that is used to define the fire resistance periods given in national regulations, so the temperatures recorded during these tests have been interpreted in terms of the standard fire resistance temperature-time curve.

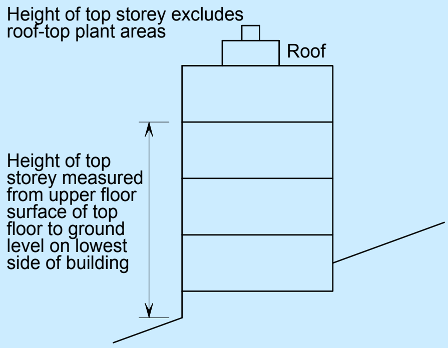
The recommended periods of fire resistance for elements of construction in various types of building in national regulations are given in Table 2.5 and Table 2.6. The structural elements of most two-storey buildings require 30 minutes fire resistance and those in most buildings between three and five storeys require 60 minutes fire resistance.

The following recommendations are for buildings in which the elements of structure are required to have up to 120 minutes fire resistance. Provided that they are followed, composite steel framed buildings will maintain their stability for this period of fire resistance, when any compartment is subject to the standard temperature-time curve [(1)].

All composite steel framed buildings with composite floors may be considered to achieve 15 minutes fire resistance without fire protection, and so no specific recommendations are given in this case.

Table 2.5 Summary of fire resistance requirements from Approved Document B for England and Wales

	Fire resistance (mins) for height of top storey (m)			
	<5	≤18	≤30	>30
Residential (non-domestic)	30	60	90	120
Office	30	60	90	120*
Shops, commercial, assembly and recreation	30	60	90	120*
Closed car parks	30	60	90	120*
Open-sided car parks	15	15	15	60



Approved Document B allows the fire resistance periods to be reduced from 60 to 30 minutes or from 90 to 60 minutes, for most purpose groups.

* Sprinklers are required, but the fire resistance of the floor may be 90 minutes only.

Table 2.6 Summary of fire resistance requirements from French Fire Regulations

<table>
<tr><td rowspan="2" colspan="2">Residential (non-domestic)</td><td>< 2 levels</td><td>2 levels < … ≤ 4 levels</td><td>4 levels < … ≤ 28 m</td><td>28 m < H < 50 m</td><td>> 50 m</td></tr>
<tr><td>R15</td><td>R30</td><td>R60</td><td>R90</td><td>R 120</td></tr>
<tr><td colspan="2"></td><td colspan="2">Ground floor</td><td>Height of the top floor ≤ 8 m</td><td>Height of the top floor > 8 m</td><td>Height of the top floor > 28 m</td></tr>
<tr><td colspan="2">Office[1]</td><td colspan="3">0</td><td>R60</td><td>R 120</td></tr>
<tr><td rowspan="3">Shops, commercial, assembly and recreation</td><td>< 100 persons</td><td colspan="3">0</td><td>R60</td><td rowspan="3">R120</td></tr>
<tr><td>< 1500 persons</td><td colspan="3">R30</td><td>R60</td></tr>
<tr><td>> 1500 persons</td><td colspan="2">R30</td><td>R60</td><td>R90</td></tr>
<tr><td colspan="2"></td><td>Ground floor</td><td>> 2 levels</td><td colspan="3">Height of the top floor > 28 m</td></tr>
<tr><td colspan="2">Closed car parks</td><td rowspan="2">R30</td><td rowspan="2">R60</td><td rowspan="2" colspan="3">R90</td></tr>
<tr><td colspan="2">Open-sided car parks</td></tr>
</table>

Note: 1. Office which is not open to the public

H is the height of the top floor

2.5.2 Natural fire (parametric temperature-time curve)

The FRACOF software allows the effect of natural fire on the floor plate to be considered using the parametric temperature-time curve as defined in EN1991-1-2 Annex A [1]. It should be noted that this is an Informative Annex and its use may not be permitted in some European countries, such as France. Before final design is undertaken the designer should consult the relevant National Annex.

Using this parametric fire curve, the software defines the compartment temperature taking account of:

- The compartment size:
  - Compartment length
  - Compartment width
  - Compartment height
- The height and area of windows:
  - Window height
  - Window length
  - Percentage open window
- The amount of combustibles and their distribution in the compartment
  - Fire Load
  - Combustion factor
  - The rate of burning
- The thermal properties of the compartment linings

The temperature of a parametric fire will often rise more quickly than the standard fire in the early stages but, as the combustibles are consumed, the temperature will decrease rapidly. The standard fire steadily increases in temperature indefinitely.

The standard temperature-time curve and a typical parametric temperature-time curve are shown in Figure 2.6.

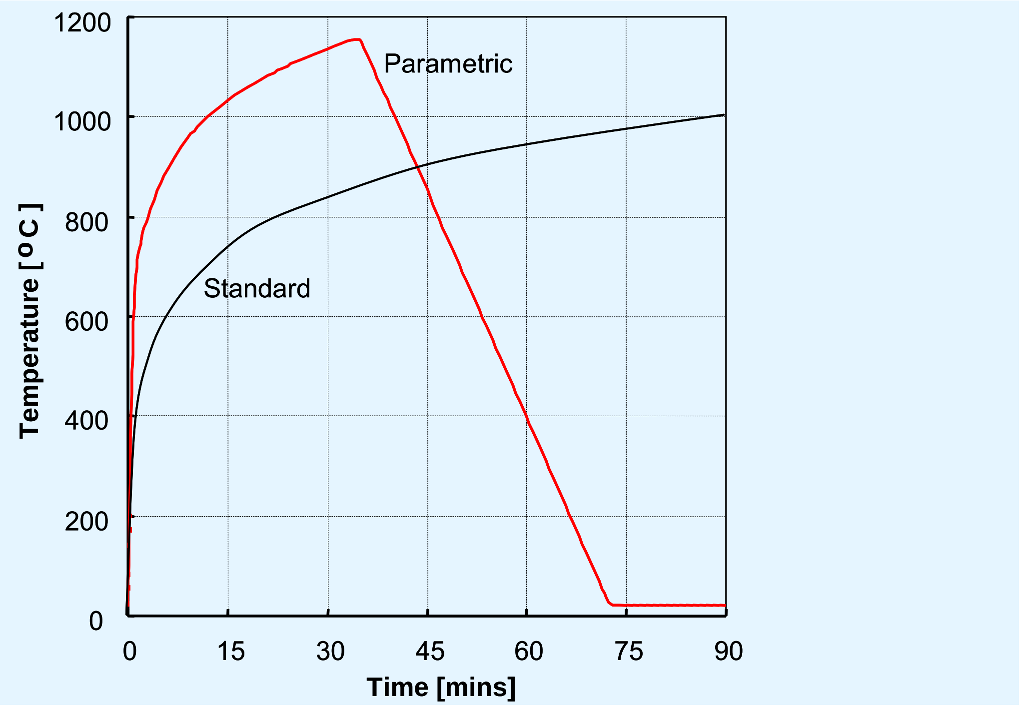


Figure 2.6 Comparison of typical parametric and standard temperature-time curve

3 RECOMMENDATIONS FOR STRUCTURAL ELEMENTS

3.1 Floor design zones

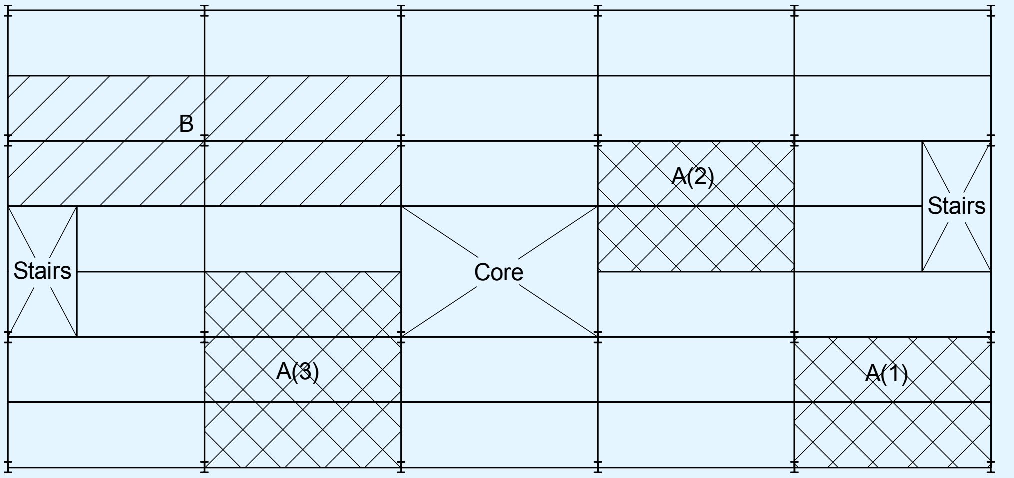
Each floor should be divided into design zones that meet the criteria given in Section 2.3.

The division of a floor into floor design zones is illustrated in Figure 3.1. Floor zones designated ‘A’ are within the scope of the FRACOF software and their load bearing performance in fire conditions may be determined using FRACOF. The zone designated ‘B’ is outside the scope of the software because it contains a column and the beams within the zone do not all span in the same direction.

A single floor zone is illustrated in Figure 3.2 showing the beam span designations used in the FRACOF software. Normal design assumes that floor loads are supported by secondary beams which are themselves supported on primary beams.

The fire design method assumes that at the fire limit state, the resistance of the unprotected internal beams reduces significantly, leaving the composite slab as a two way spanning element simply supported around its perimeter. In order to ensure that the slab can develop membrane action, the FRACOF software computes the moment applied to each perimeter beam as a result of the actions on the floor design zone. To maintain the vertical support to the perimeter of the floor design zone in practice, the software calculates the degree of utilisation and hence the critical temperature of these perimeter beams. The fire protection for these beams should be designed on the basis of this critical temperature and the fire resistance period required for the floor plate in accordance with national regulations. The critical temperature and the degree of utilisation for each perimeter beam is reported for Side A to D of the floor design zone as shown by Figure 3.2.

As noted in Section 2.2.2, a restriction on the use of the FRACOF software is that for 60 minutes or more fire resistance, the zone boundaries should align with the column grid and the boundary beams should be fire protected. For 30 minutes fire resistance, this restriction does not apply and the zone boundaries do not have to align with the column grid. For example, in Table 3.1, zones A2 and A3 have columns at only two of their corners and could only be considered as design zones for a floor that requires no more than 30 minutes fire resistance.



Key to figure

A: These zones may be designed using FRACOF

A(1) Any period of fire resistance

A(2) & A(3) only 30 minutes fire resistance

B: Outside the scope of FRACOF

Figure 3.1 Possible floor design zones

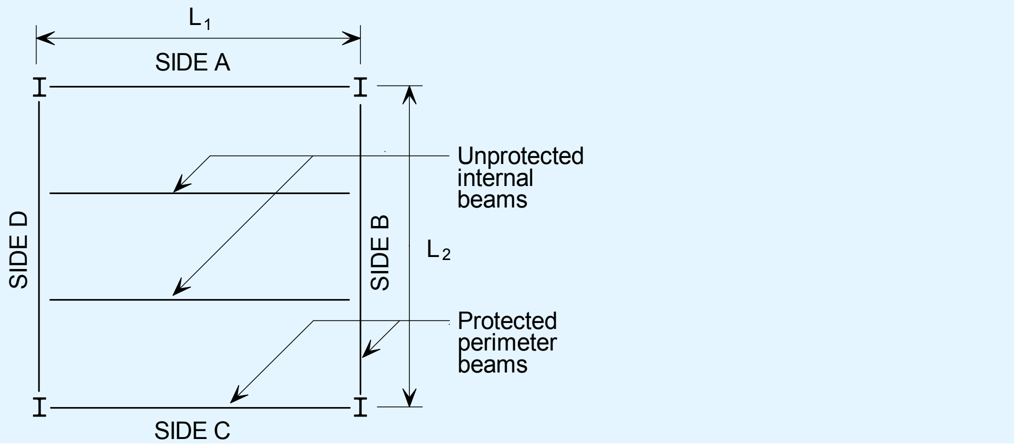


Figure 3.2 Definition of span 1 (L_1) and span 2 (L_2) and the beam layout for a floor design zone in a building requiring fire resistance of 60 minutes or more.

3.2 Floor slab and beams

The FRACOF software calculates the load bearing capacity of the floor slab and unprotected beams at the fire limit state. As the simple design method, implemented in the software, assumes that the slab will have adequate support on its perimeter the software also calculates the critical temperature for each perimeter beam based on the load bearing capacity of the floor design zone.

3.2.1 Fire design of floor slab

Load bearing performance of the composite floor slab

When calculating the load bearing capacity of each floor design zone the resistance of the composite slab and the unprotected beams are calculated separately. The slab is assumed to have no continuity along the perimeter of the floor design zone. The load that can be supported by the flexural behaviour of the composite slab within the floor design zone is calculated based on a lower bound mechanism assuming a yield line pattern as shown in Figure 3.3.

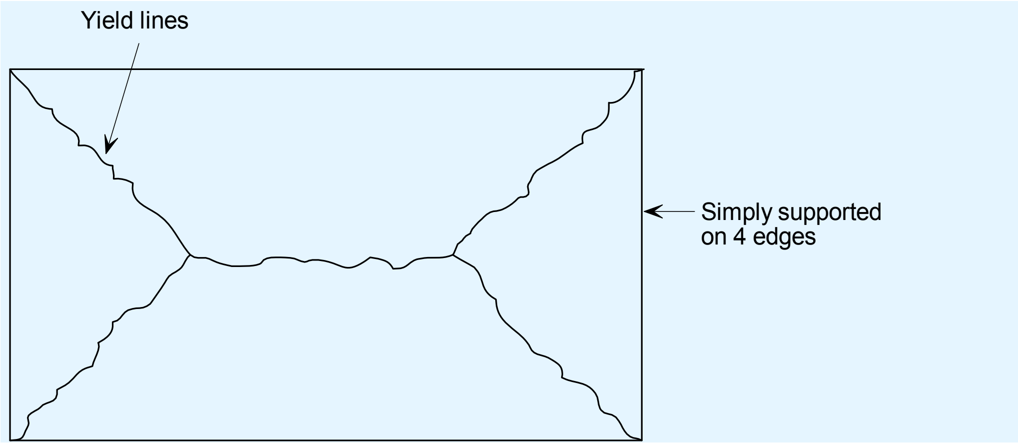


Figure 3.3 Assumed yield line pattern used to calculate slab resistance

The value of the resistance calculated using the lower bound mechanism is enhanced by considering the beneficial effect of tensile membrane action at large displacements. This enhancement increases with increasing vertical deflection of the slab until failure occurs due to fracture of the reinforcement across the short slab span or compressive failure of the concrete in the corners of the slab, as shown by Figure 3.4. As the design method can not predict the point of failure, the value of deflection considered when calculating the enhancement is based on a conservative estimate of slab deflection that includes allowance for the thermal curvature of the slab and the strain in the reinforcement, as shown below.

$$w = \frac{\alpha (T_2 - T_1) l^2}{19.2h} + \sqrt{\left(\frac{0.5 f_y}{E_a}\right)\frac{3L^2}{8}}$$

The deflection allowed due to elongation of the reinforcement is also limited by the following expression.

$$w \le \frac{\alpha (T_2 - T_1) l^2}{19.2h} + \frac{l}{30}$$

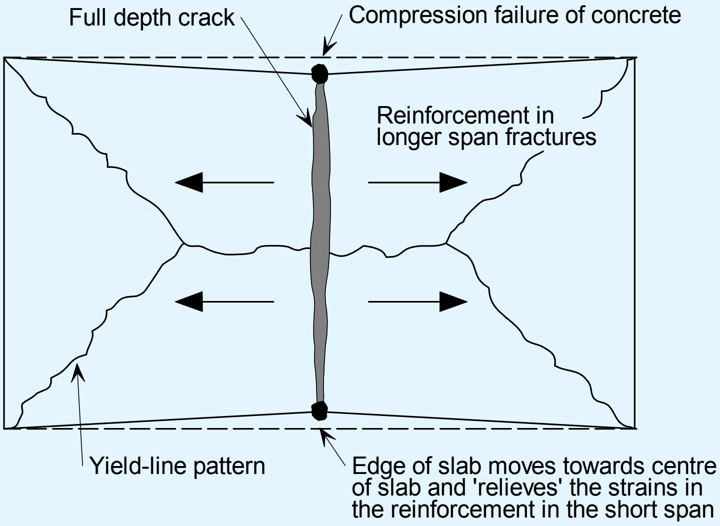
Where

$(T_2 - T_1)$	is the temperature difference between the top and bottom surface of the slab
L	is the longer dimension of the floor design zone
l	is the shorter dimension of the floor design zone
f_y	is the yield strength of the mesh reinforcement
E	is the modulus of elasticity of the steel
h	is the overall depth of the composite slab
α	is the coefficient of thermal expansion of concrete

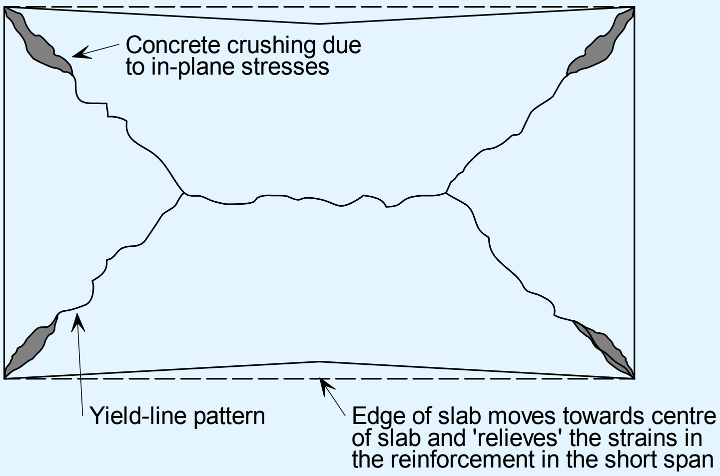
All of the available test evidence shows that this value of deflection will be exceeded before load bearing failure of the slab occurs. This implies that the resistance predicted using the design method will be conservative compared to its actual performance.

The overall deflection of the slab is also limited by the following expression.

$$w \leq \frac{L+l}{30}$$



(a) Tensile failure of the reinforcement



(b) Compressive failure of the concrete

Figure 3.4 Failure mode due to fracture of the reinforcement

The residual bending resistance of the unprotected composite beams is then added to the enhanced slab resistance to give the total resistance of the complete system.

Integrity and insulation performance of the composite slab

The FRACOF software does not explicitly check the insulation or integrity performance of the floor slab. The designer must therefore ensure that the slab thickness chosen is sufficient to provide the necessary insulation performance in accordance with the recommendations given in EN 1994-1-2.

To ensure that the composite slab maintains its integrity during the fire and that membrane action can develop, care must be taken to ensure that the reinforcing mesh is properly lapped. This is especially important in the region of unprotected beams and around columns. Further information on required lap lengths and placement of the reinforcing mesh is given in Section 3.3.

3.2.2 Fire design of beams on the perimeter of the floor design zone.

The beams along the perimeter of the floor design zone, labelled A to D in Figure 3.2, should achieve the fire resistance required for the floor plate, in order to provide the required vertical support to the perimeter of the floor design zone. This usually results in these beams being fire protected.

The FRACOF software calculates the design effect of actions on these perimeter beams and the room temperature moment of resistance of the beam, in order to calculate the degree of utilisation for each perimeter beam, which is calculated using the guidance given in EN 1993-1-2 §4.2.4, as shown below.

$$\mu_0 = \frac{E_{\mathrm{fi,d}}}{R_{\mathrm{fi,d,0}}}$$

Where

$E_{\mathrm{fi,d}}$ is the design effect of actions on the beam in fire

$R_{\mathrm{fi,d,0}}$ is the design resistance of the beam at time t = 0

Having calculated the degree of utilisation, the software can compute the critical temperature of the bottom flange of the perimeter beams. This critical temperature is reported in the FRACOF software output for use when specifying the fire protection required by each of the perimeter beams on the floor design zone. Full details of the calculation method can be obtained from the FRACOF Engineering Background(7).

For perimeter beams with floor design zones on both sides, the lower value of critical temperature given by the design of the adjacent floor design zones should be used to design the fire protection for that perimeter beam. The method of design for a perimeter beam that is shared by two floor design zones is illustrated in the work example, see Section 5.3.1.

When specifying fire protection for the perimeter beams, the fire protection supplier must be given the section factor for the member to be protected and the period of fire resistance required and the critical temperature of the member. Most reputable fire protection manufacturers will have a multi temperature assessment for their product which will have been assessed in accordance with EN 13381-4(17) for non-reactive materials or EN 13381-8(18) for reactive materials (intumescents). Design tables for fire protection which relate section factor to protection thickness are based on a single value of assessment temperature. This assessment temperature should be less than or equal to the critical temperature of the member.

3.3 Reinforcement details

The yield strength and ductility of the reinforcing steel material should specified in accordance with the requirements of EN 10080. The characteristic yield strength of reinforcement to EN 10080 will be between 400 MPa and

600 MPa, depending on the national market. In order that the reinforcement has sufficient ductility to allow the development of tensile membrane action, Class B or Class C should be specified

In most countries, national standards for the specification of reinforcement may still exist as non-contradictory complimentary information (NCCI), as a common range of steel grades have not been agreed for EN 10080.

In composite slabs, the primary function of the mesh reinforcement is to control the cracking of the concrete. Therefore the mesh reinforcement tends to be located as close as possible to the surface of the concrete while maintaining the minimum depth of concrete cover required to provide adequate durability, in accordance with EN 1992-1-1[19]. In fire conditions, the position of the mesh will affect the mesh temperature and the lever arm when calculating the bending resistance. Typically, adequate fire performance is achieved with the mesh placed between 15 mm and 45 mm below the top surface of the concrete.

Section 3.3.1 gives general information regarding reinforcement details. Further guidance and information can be obtained from, EN 1994-1-1(9) and EN 1994-1-2[6] or any national specifications such as those given in reference (20)

3.3.1 Detailing mesh reinforcement

Typically, sheets of mesh reinforcement are 4.8 m by 2.4 m and therefore must be lapped to achieve continuity of the reinforcement. Sufficient lap lengths must therefore be specified and adequate site control must be put in place to ensure that such details are implemented on site. Recommended lap lengths are given in section 8.7.5 of EN1992-1-1[19] or can be in accordance with Table 3.1. The minimum lap length for mesh reinforcement should be 250 mm. Ideally, mesh should be specified with 'flying ends', as shown in Figure 3.5, to eliminate build up of bars at laps. It will often be economic to order 'ready fit fabric', to reduce wastage.

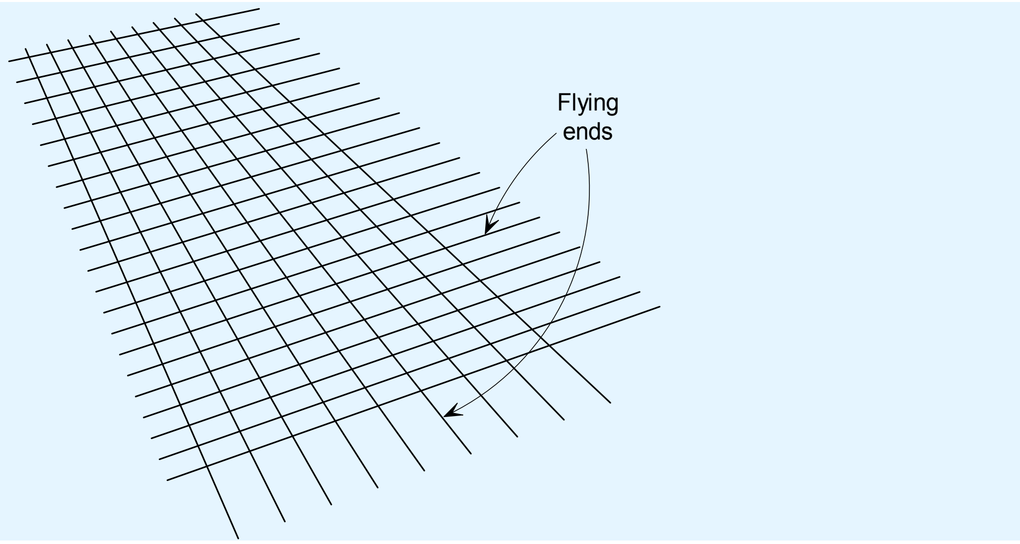


Figure 3.5 Mesh with flying ends

Table 3.1 Recommended tension laps and anchorage lengths for welded mesh

Reinforcement Type	Wire/Bar Type	Concrete Grade					
		LC 25/28	NC 25/30	LC 28/31	NC 28/35	LC 32/35	NC 32/40
Grade 500 Bar of diameter d	Ribbed	50d	40d	47d	38d	44d	35d
6 mm wires							
	Ribbed	300	250	300	250	275	250
7 mm wires							
	Ribbed	350	300	350	275	325	250
8 mm wires							
	Ribbed	400	325	400	325	350	300
10 mm wires							
	Ribbed	500	400	475	400	450	350

Notes:

These recommendations can be conservatively applied to design in accordance with EN 1992-1-1.

Where a lap occurs at the top of a section and the minimum cover is less than twice the size of the lapped reinforcement, the lap length should be increased by a factor of 1.4.

Ribbed Bars/Wires are defined in EN 10080

The minimum Lap/Anchorage length for bars and fabric should be 300 mm and 250 mm respectively.

3.3.2 Detailing requirements for the edge of a composite floor slab

The detailing of reinforcement at the edge of the composite floor slab will have a significant effect on the performance of the edge beams and the floor slab in fire conditions. The following guidance is based on the best practice recommendations for the design and construction of composite floor slabs to meet the requirements for room temperature design. The fire design method and guidance presented in this document assumes that the composite floor is constructed in accordance with these recommendations.

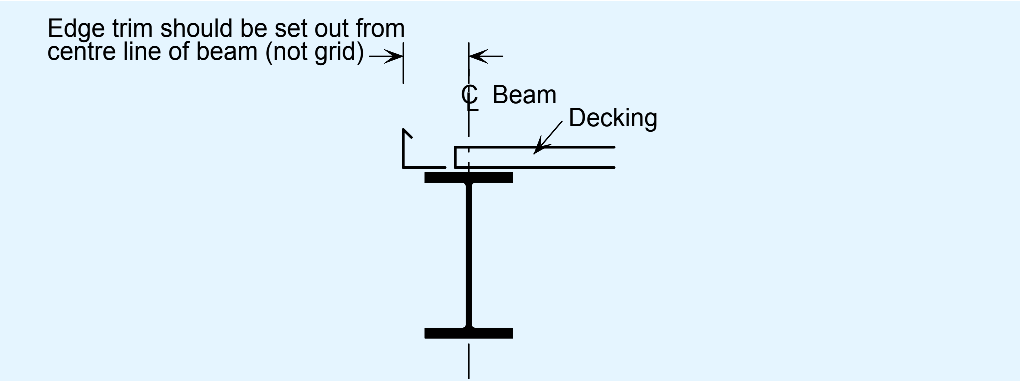


Figure 3.6 Setting out of edge trim

The edge of the composite slab is usually formed using ‘edge trims’ made from strips of light gauge galvanized steel fixed to the beam in the same way as the decking, as shown in Figure 3.6. In cases where the edge beam is designed to act compositely with the concrete slab, U shaped reinforcing bars are required to prevent longitudinal splitting of the concrete slab. These reinforcement bars also ensure that the edge beam is adequately anchored to the slab when using this simple design method.

Some typical slab edge details covering the two deck orientations are given in Figure 3.7. Where the decking ribs run transversely over the edge beam and cantilevers out a short distance, the edge trim can be fastened in the manner suggested in Figure 3.7 (a). The cantilever projection should be no more than 600 mm, depending on the depth of the slab and deck type used.

The more difficult case is where the decking ribs run parallel to the edge beam, and the finished slab is required to project a short distance, so making the longitudinal edge of the sheet unsupported Figure 3.7 (b). When the slab projection is more than approximately 200 mm (depending on the specific details), the edge trim should span between stub beams attached to the edge beam, as shown in Figure 3.7 (c). These stub beams are usually less than 3 m apart, and should be designed and specified by the structural designer as part of the steelwork package.'

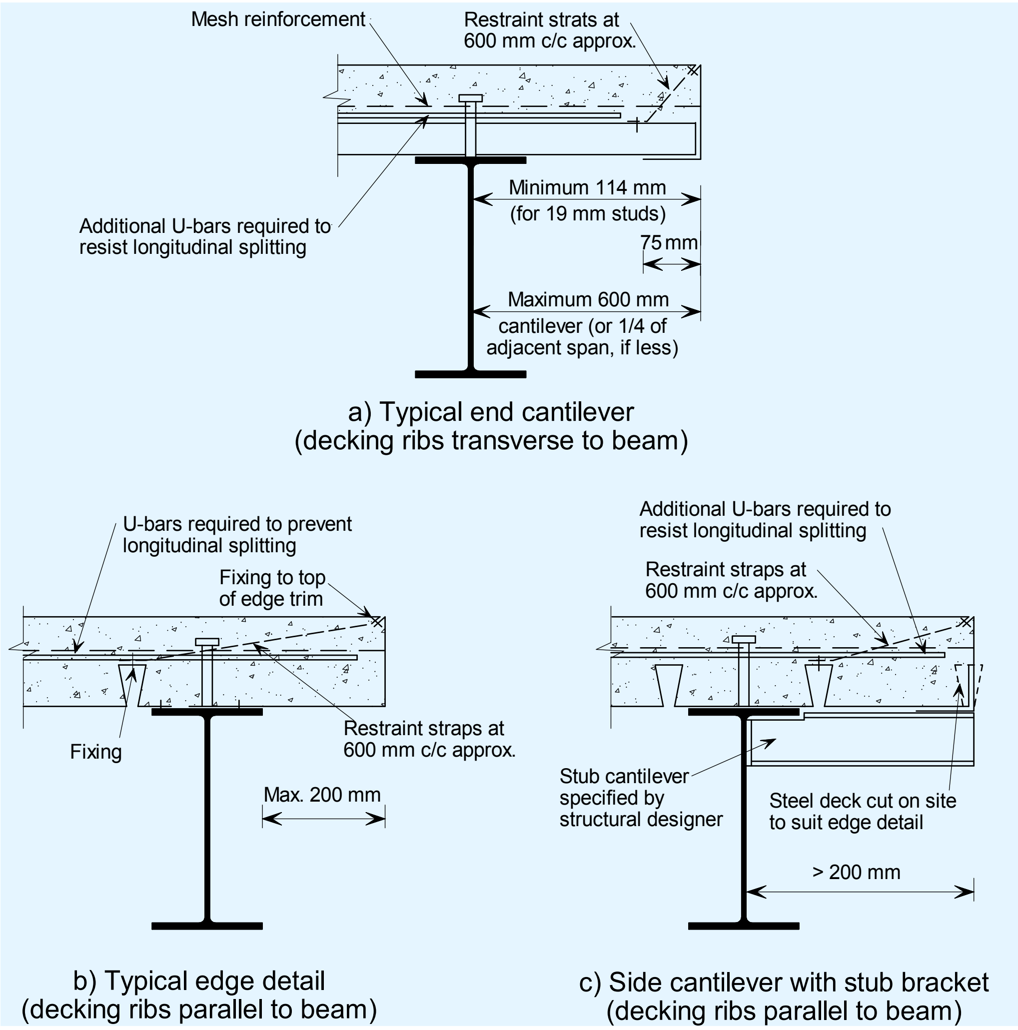


Figure 3.7 Typical edge details

3.4 Design of non composite edge beams

It is common practice for beams at the edge of floor slabs to be designed as non composite beams. This is because the costs of meeting the requirements for transverse shear reinforcement are more than the costs of installing a slightly heavier non composite beam. For fire design, it is important that the floor slab is adequately anchored to the edge beams, as these beams will be at the edge of floor design zones. Although not usually required for room temperature design of non composite edge beams, this guide recommends that shear connectors are provided at not more than 300 mm centres and U shaped reinforcing bars positioned around the shear connectors, as described in Section 3.3.2.

Edge beams often serve the dual function of supporting both the floors and the cladding. It is important that the deformation of edge beams should not affect the stability of cladding as it might increase the danger to fire fighters and others in the vicinity. (This does not refer to the hazard from falling glass that results from thermal shock, which can only be addressed by use of special materials or sprinklers.) Excessive deformation of the façade could increase the hazard, particularly when a building is tall and clad in masonry, by causing bricks to be dislodged.

3.5 Columns

The design guidance in this document is devised to confine structural damage and fire spread to the fire compartment itself. In order to achieve this, columns (other than those in the top storey) should be designed for the required period of fire resistance or designed to withstand the selected natural (parametric) fire.

Any applied fire protection should extend over the full height of the column, including the connection zone (see Figure 3.8). This will ensure that no local squashing of the column occurs and that structural damage is confined to one floor.

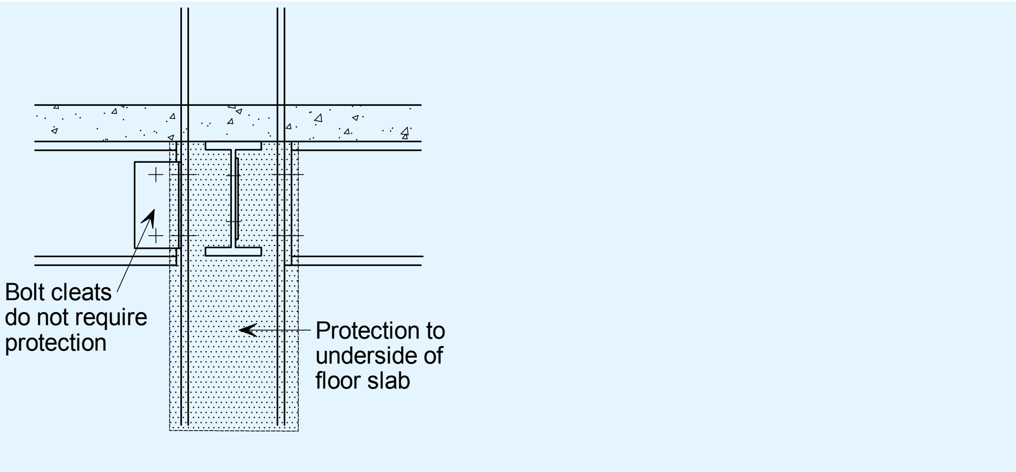


Figure 3.8 Extent of fire protection to columns

In the Cardington fire tests, the protected columns performed well with no sign of collapse. However, subsequent finite element modelling has indicated the possibility that premature column failure could occur in some circumstances. A mode of behaviour has been identified [22] in which expansion of the floors induces moments in the columns. This can have the effect of reducing the temperature at which a column would fail.

It is recommended that, as a conservative measure, the protection to the columns at the edge of the floor plate in buildings of more than two storeys should be increased by basing its thickness on a critical temperature of 500°C, or 80ºC less than the critical temperature given in EN 1993-1-2, whichever is the lower.

For most board fire protection materials, this reduction in critical temperature will have no effect, as the minimum available thickness of board will suffice.

3.6 Joints

As stated in Section 2.2.1 the values given by the design method relate to 'simple' joints such as those with flexible end plates, fin plates and web cleats.

The steel frame building tested at Cardington contained flexible end plate and fin plate connections. Partial and full failures of some of the joints were observed during the cooling phase of the Cardington fire tests; however, no failure of the structure occurred as a result.

In the case where the plate was torn off the end of the beam, no collapse occurred because the floor slab transferred the shear to other load paths. This highlights the important role of the composite floor slab, which can be achieved with proper lapping of the reinforcement.

The resistances of the simple joints should be verified using the rules given in EN 1993-1-8 [(23)].

3.6.1 Joint classification

Joint details should be such that they fulfill the assumptions made in the design model. Three joint classifications are given in EN 1993-1-8:

- Nominally pinned
  - Joints that transfer internal shear forces without transferring significant moments.
- Semi-rigid
  - Joints that do not satisfy the nominally pinned nor the rigid joint criteria.
- Rigid
  - Joints that provide full continuity.

EN 1993-1-8 §5.2 gives principles for the classification of joints based on their stiffness and strength; the rotation capacity (ductility) of the joint should also be considered.

As stated in Section 2.2.1 the values given by the simple design method have been prepared assuming the use of nominally pinned (simple) joints. To ensure that a joint does not transfer significant bending moments and so that it is a ‘simple’ joint it must have sufficient ductility to allow a degree of rotation. This can be achieved by detailing the joint such that it meets geometrical limits. Guidance on geometrical limits and initial sizing to ensure sufficient ductility of the joint is given in Access-steel documents (25).

3.6.2 End plates

There are two basic types of end plate connections; partial depth; and full depth. SN013 recommends the use of:

Partial end plates when; $V_{Ed} \leq 0.75\ V_{c,Rd}$

Full depth end plates when; $0.75\ V_{c,Rd} < V_{Ed} \leq V_{c,Rd}$

Where:

V_{Ed} is the design shear force applied to the joint

$V_{c,Rd}$ is the design shear resistance of the supported beam.

The resistance of the components of the joint should be verified against the requirements given in EN 1993-1-8. For persistent and transient design situations the following design resistances need to be verified at ambient temperatures:

- End plate bolt group *
- Supporting member in bearing
- End plate in shear (gross section)
- End plate in shear (net section)
- End plate in shear (block shear)
- End plate in bending
- Beam web in shear*

For completeness, all the design verifications given above should be carried out. However, in practice, for ‘normal’ joints, the verifications marked * will usually be critical. Guidance on meeting the requirements of EN 1993-1-8 is given in Access-steel documents(26).

EN 1993-1-8 does not give any guidance on design for tying resistance of end plates. Guidance is given in SN015[26] for the determination of the tying resistance of an end plate.

3.6.3 Fin plates

Single and double vertical lines of bolts may be used in fin plates. SN014(26) recommends the use of:

Single vertical lines of bolts when; $V_{Ed} \leq 0.50\ V_{c,Rd}$

Two vertical lines of bolts when; $0.50\ V_{c,Rd} < V_{Ed} \leq 0.75\ V_{c,Rd}$

Use an end plate when; $0.75\ V_{c,Rd} < V_{Ed}$

Where:

V_{Ed} is the design shear force applied to the joint

$V_{c,Rd}$ is the design shear resistance of the supported beam.

For persistent and transient design situations, the following fin plate design resistances need to be verified at ambient temperature:

- Bolts in shear*
- Fin plate in bearing*
- Fin plate in shear (gross section)
- Fin plate in shear (net section)
- Fin plate in shear (block shear)
- Fin plate in bending
- Fin plate in buckling (LTB)
- Beam web in bearing*
- Beam web in shear (gross section)
- Beam web in shear (net section)
- Beam web in shear (block shear)
- Supporting element (punching shear) (This mode is not appropriate for fin plates connected to column flanges)

For completeness, all the design verifications given above should be carried out. However, in practice, for 'normal' joints, the verifications marked * will usually be critical. Guidance on meeting the requirements of EN 1993-1-8 is given in Access Steel documents[27].

As for end plates EN 1993-1-8 does not give any guidance on design for tying resistance of fin plates. Therefore, alternative guidance such as that given in SN018[27] may be used to determine the tying resistance of a fin plate.

3.6.4 Web cleats

Although there were no cleated joints used in the Cardington frame, SCI has conducted a number of tests on composite and non-composite cleated joints in fire (28). These joints consisted of two steel angles bolted to either side of the beam web using two bolts in each angle leg, then attached to the flange of the column also using two bolts. The joints were found to be rotationally ductile under fire conditions and large rotations occurred. This ductility was due to plastic hinges that formed in the leg of the angle adjacent to the column face. No failure of bolts occurred during the fire test. The composite cleated joint had a better performance in fire than the non-composite joint.

For non-composite web cleat joints it is recommended that single vertical lines of bolts should only be used when:

$V_{Ed} \leq 0.50\ V_{c,Rd}$

The design resistance of the cleated joint should be verified using the design rules given in Section 3 of EN 1993-1-8. Table 3.3 of EN 1993-1-8 gives the maximum and minimum values for the edge, end and spacing distances that should be met when detailing the position of bolts.

3.6.5 Fire protection

In cases where both structural elements to be connected are fire protected, the protection appropriate to each element should be applied to the parts of the plates or angles in contact with that element. If only one element requires fire protection, the plates or angles in contact with the unprotected elements may be left unprotected.

3.7 Overall building stability

In order to avoid sway collapse, the building should be braced by shear walls or other bracing systems. Masonry or reinforced concrete shear walls should be constructed with the appropriate fire resistance.

If bracing plays a major part in maintaining the overall stability of the building it should be protected to the appropriate standard.

In two-storey buildings, it may be possible to ensure overall stability without requiring fire resistance for all parts of the bracing system. In taller buildings, all parts of the bracing system should be appropriately fire protected.

One way in which fire resistance can be achieved without applied protection is to locate the bracing system in a protected shaft such as a stairwell, lift shaft or service core. It is important that the walls enclosing such shafts have adequate fire resistance to prevent the spread of any fire. Steel beams, columns and bracing totally contained within the shaft may be unprotected. Other steelwork supporting the walls of such shafts should have the appropriate fire resistance.

4 COMPARTMENTATION

National regulations require that compartment walls separating one fire compartment from another shall have stability, integrity and insulation for the required fire resistance period.

Stability is the ability of a wall not to collapse. For loadbearing walls, the loadbearing capacity must be maintained.

Integrity is the ability to resist the penetration of flames and hot gases.

Insulation is the ability to resist excessive transfer of heat from the side exposed to fire to the unexposed side.

4.1 Beams above fire resistant walls

When a beam is part of a fire resisting wall, the combined wall/beam separating element must have adequate insulation and integrity as well as stability. For optimum fire performance, compartment walls should, whenever possible, be located beneath and in line with beams.

Beams in the wall plane

The Cardington tests demonstrated that unprotected beams above and in the same plane as separating walls (see Figure 4.1), which are heated from one side only, do not deflect to a degree that would compromise compartment integrity, and normal movement allowances are sufficient. Insulation requirements must be fulfilled and protection for 30 or 60 minutes will be necessary; all voids and service penetrations must be fire stopped. Beams protected with intumescent coatings require additional insulation because the temperature on the non fire side is likely to exceed the limits required in the fire resistance testing standards [29,30].

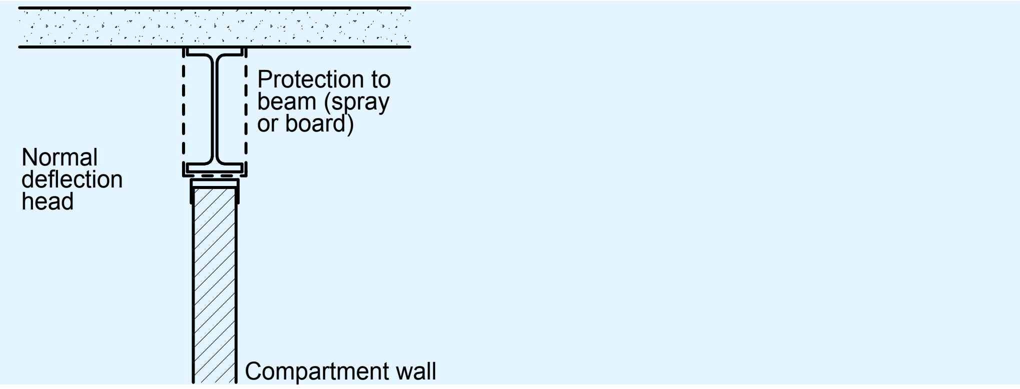


Figure 4.1 Beams above and in line with walls

Beams through walls

The Cardington tests showed that floor stability can be maintained even when unprotected beams suffer large deflections. However, when walls are located off the column grid, large deflections of unprotected beams can compromise integrity by displacing or cracking the walls through which they pass. In such cases, the beams should either be protected or sufficient movement allowance provided. It is recommended that a deflection allowance of span/30 should be provided in walls crossing the middle half of an unprotected beam. For walls crossing the end quarters of the beam, this allowance may be reduced linearly to zero at end supports (see Figure 4.2). The compartment wall should extend to the underside of the floor.

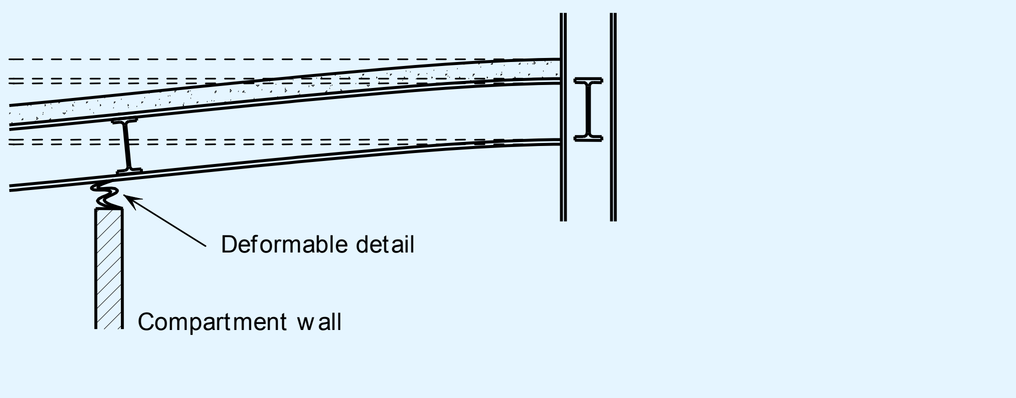


Figure 4.2 Deformation of beams crossing walls

4.2 Stability

Walls that divide a storey into more than one fire compartment must be designed to accommodate expected structural movements without collapse (stability). Where beams span above and in the plane of the wall, movements, even of unprotected beams, may be small and the normal allowance for deflection should be adequate. If a wall is not located at a beam position, the floor deflection that the wall will be required to accommodate may be large. It is therefore recommended that fire compartment walls should be located at a beam positions whenever possible.

In some cases, the deflection allowance may be in the form of a sliding joint. In other cases, the potential deflection may be too large and some form of deformable blanket or curtain may be required, as illustrated in Figure 4.2.

National recommendations should be consulted for the structural deformations which should be considered when ensuring that compartmentation is maintained.

4.3 Integrity and insulation

Steel beams above fire compartment walls are part of the wall and are required to have the same separating characteristics as the wall. A steel beam without penetrations will have integrity. However, any service penetrations must be properly fire stopped and all voids above composite beams should also be fire stopped.

An unprotected beam in the plane of a compartment wall may not have the required insulation and will normally require applied fire protection. It is recommended that all beams at compartment boundaries should be fire protected, as shown in Figure 4.1.

5 WORKED EXAMPLE

In order to illustrate the application of the output from the FRACOF software, this Section contains a worked example based on a realistic composite floor plate.

The building considered is a 4 storey steel framed office building. The building requires 60 minutes fire resistance in accordance with the requirements of National Building Regulations.

The floor plate for each storey consists of a composite floor slab constructed using Cofraplus 60 trapezoidal metal decking, normal weight concrete and a single layer of mesh reinforcement. The slab spans between 9 m long secondary beams designed to act compositely with the floor slab. These secondary beams are also in turn supported on composite primary beams of 9 m and 12 m spans. The beams on the edge of the building are designed as non-composite in accordance with EN 1993-1-1.

The construction of the floor plate is shown in Figure 5.1 to Figure 5.4.

Figure 5.1 shows the general arrangement of steelwork at floor level across the full width of the building and two bays along its length. It is assumed that this general arrangement is repeated in adjoining bays along the length of the building. The columns are HD 320 x 158, designed as non-composite columns in accordance with EN 1993-1-1.

The floor loading considered was as follows

- Variable action due to occupancy: 4 kN/m^2
- Variable action due to light weight partitions: 1 kN/m^2
- Permanent action due to ceilings and services: 0.7 kN/m^2
- Self weight of beam: 0.5 kN/m^2

For the edge beams, an additional cladding load of 2 kN/m was considered in their design.

The beam sizes required to fulfil the normal stage checks for these values of actions are shown in Figure 5.1. The internal beams are composite and the degree of shear connection for each beam is shown in

Table 5.1.

Figure 5.2 shows a cross section through the composite slab. The slab is C25/30 normal weight concrete with overall thickness of 130 mm. The slab is reinforced with ST 15C mesh reinforcement with a yield strength of 500 MPa, this meets the requirements for normal temperature design but the mesh size may need to be increased in size if the performance in fire conditions is inadequate.

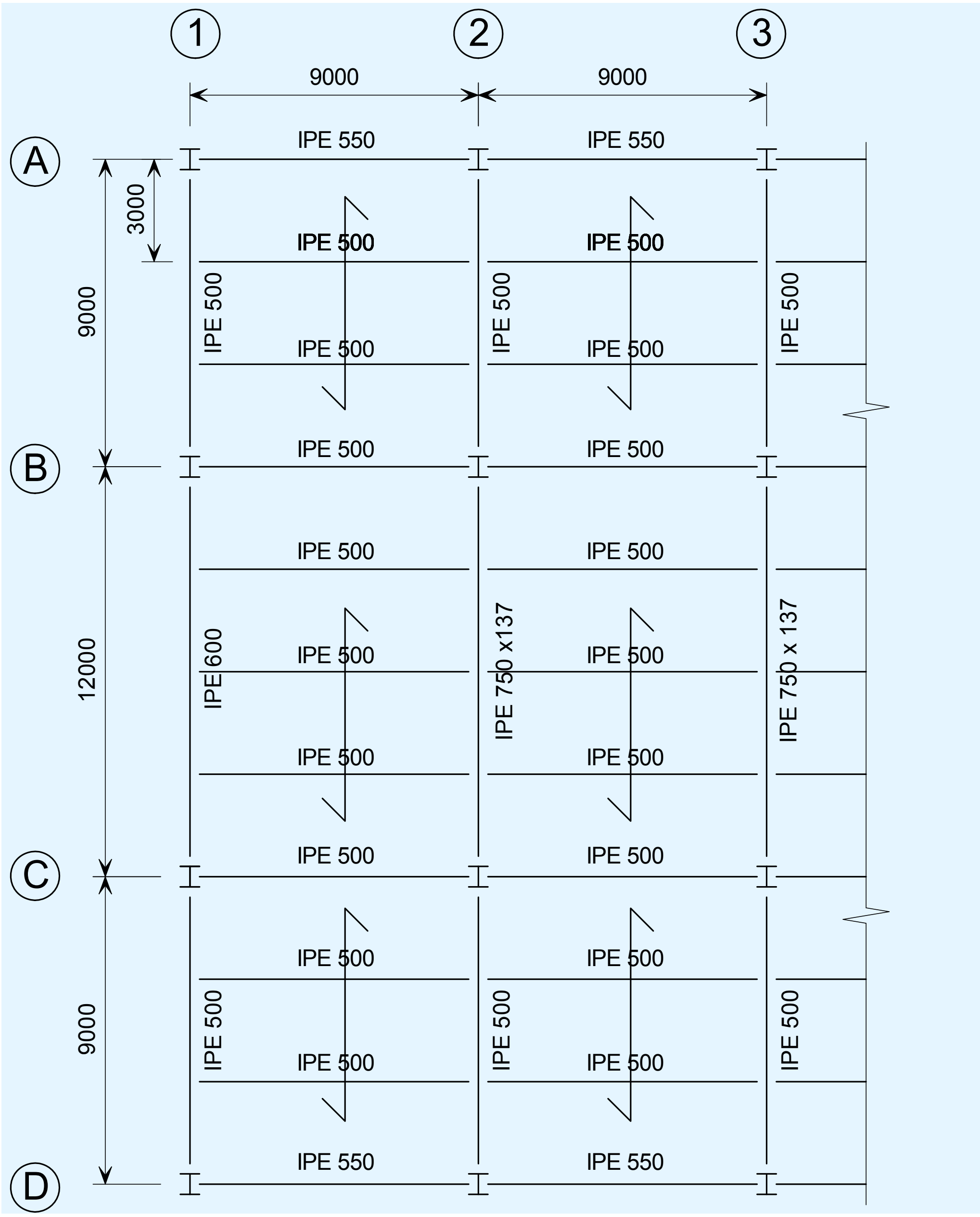


Figure 5.1 General arrangement of steelwork at floor level

Table 5.1 Beam details

Beam Section (S355)	Location of beam	Construction Type	Degree of Shear Connection (%)	Number of shear studs per group and spacing
IPE 500	Secondary internal beam	Composite	51	1 @ 207mm
IPE 550	Secondary edge beam	Non composite	N/A	
IPE 500	Primary internal beam	Composite	72	2 @ 207mm
IPE 500	Primary edge beam	Non composite	N/A	
IPE 750 × 137	Primary internal beam	Composite	71	2 @ 207 mm
IPE 600	Primary edge beam	Non Composite	N/A	

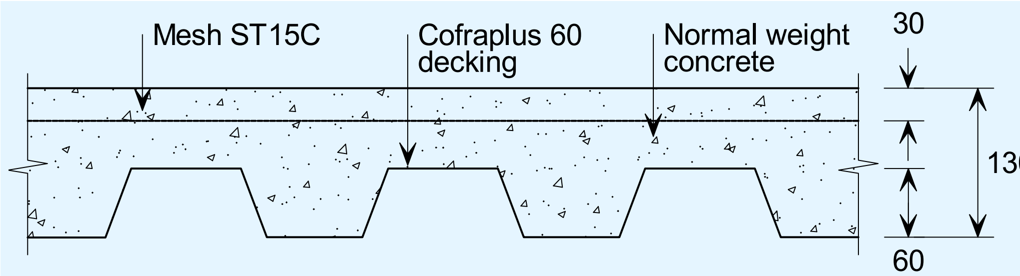
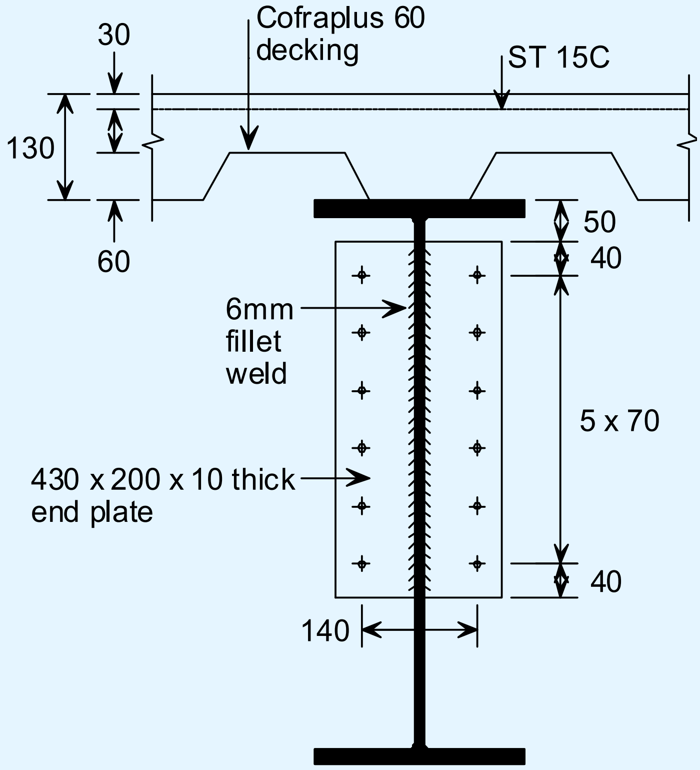
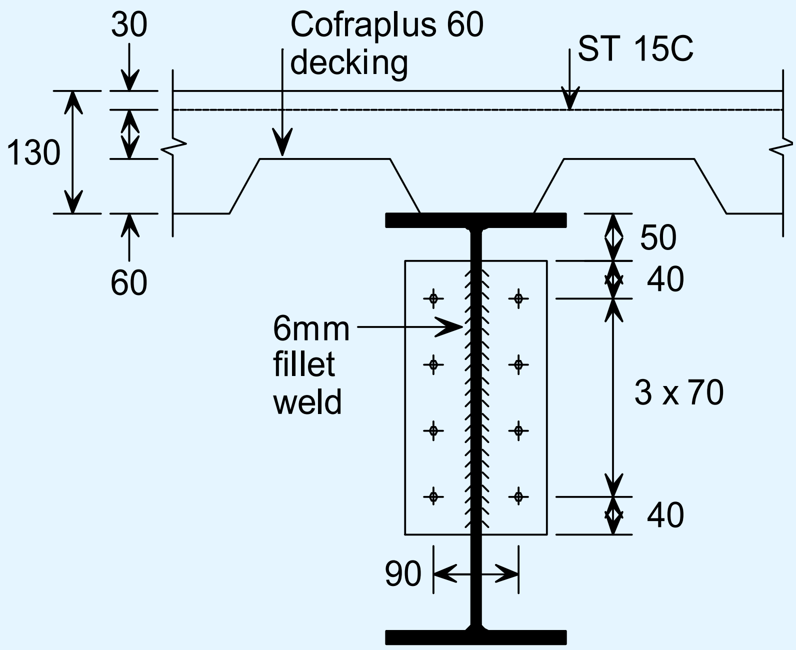


Figure 5.2 Construction of floor slab

All joints between the main steelwork elements use flexible end plate details and are designed as nominally pinned in accordance with EN1993-1-8. Figure 5.3(a) shows the joint used between the primary beams and the columns. The beam to column joints for secondary beams are as shown in Figure 5.3(b). Figure 5.4 shows the endplate connection between the secondary beams and the primary beams.



(a) Primary beam to column joint



(b) Secondary beam to column joint

Figure 5.3 Beam to column joints.

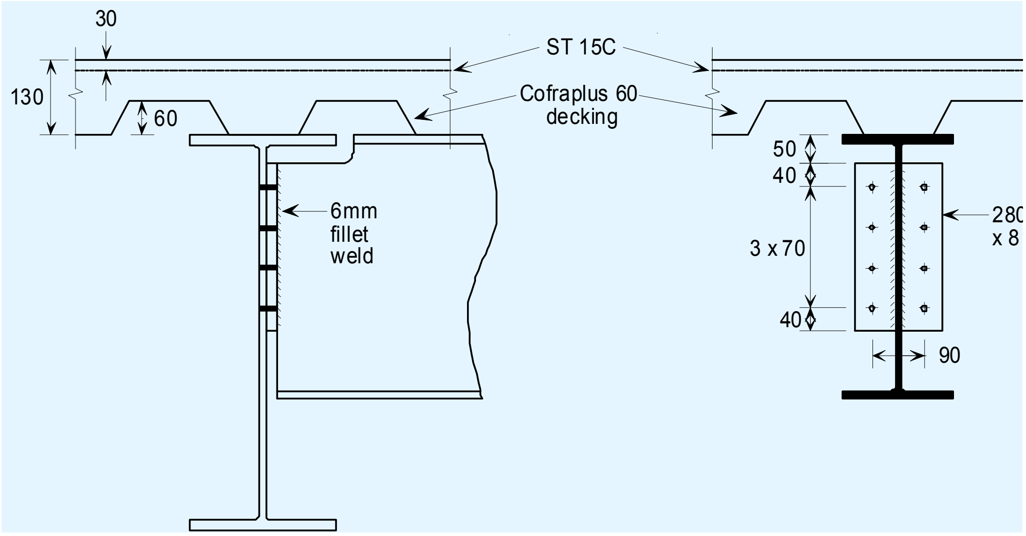


Figure 5.4 Secondary beam to primary beam connection

Figure 5.5 shows the floor plate divided into floor design zones. It is likely that floor design zones A and B will give the most onerous design conditions. The design of both of these zones will be considered.

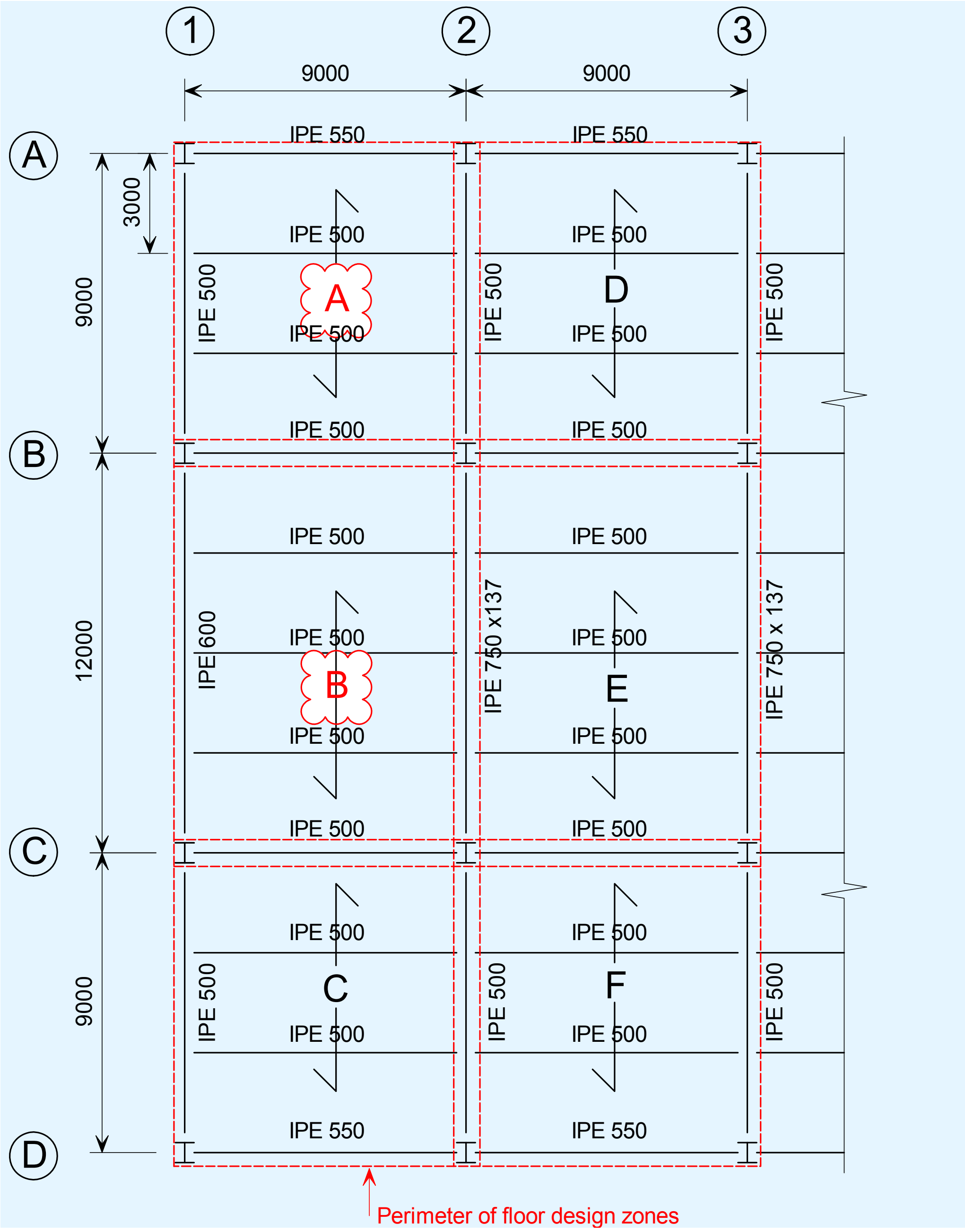


Figure 5.5 Floor design zones (A – F)

5.1 Design of composite slab in fire conditions

The following design checks carried out on the floor design zones are based on the floor construction required for room temperature design checks. If this construction proves to be inadequate for fire conditions then the mesh size and/or the floor depth will be increased to improve the performance in fire conditions. As the design zone B seems more critical than design zone A due to its lager span, we run the program with design zone B first.

5.1.1 Floor design zone B

Figure 5.6to Figure 5.8 shows the input and output from the FRACOF software for floor design zone B, which is 9 m by 12 m with the mesh size of ST 15C, Within this floor design zone, there are 3 unprotected composite beams.

From the output, the load bearing capacity of the slab based on the lower bound yield line mechanism is seen to be 0.46 kN/m^2. This capacity is enhanced due to the effect of membrane action to give a slab capacity of 2.83 kN/m^2 at 60 minutes. The enhancement factor at 60 minutes was based on a slab deflection of 629 mm.

The load bearing capacity of the composite beams is added to the slab capacity to given the total load bearing capacity. The beam capacity is based on the temperature of the unprotected beams at each time step. At 60 minutes, the bending resistance of the three unprotected beams is 2.56 kN/m^2. Thus, the total load bearing resistance of the floor design zone is 2.83 + 2.56= 5.39 kN/m^2, which is less than the applied load 6.35 kN/m^2. The size of the reinforcing mesh must therefore be increased in order to satisfy the fire requirements.

- Spans

Span 1:	9 m
Span 2:	12 m

- Unprotected Beams

Number of internal unprotected beams:	3

3. Deck Details

- Deck Properties

Deck:	COFRAPLUS 60	Type:	Trapezoidal
Depth:	58 mm	Top flange:	106 mm
Pitch:	207 mm	Bottom flange:	62 mm
Stiffener height:	0 mm		

4. Slab Details

- Concrete

Concrete type:	Normal	Slab depth:	130 mm
		Cylinder compressive strength of concrete (f_{ck}):	25 N/mm²

- Mesh

Mesh type:	ST 15 C		
Transverse mesh area:	142 mm²/m	Bar size:	6 mm
Longitudinal mesh area:	142 mm²/m	Bar size:	6 mm
Average mesh axis distance:	30 mm	Mesh yield stress:	500 N/mm²

5. Beams Details

- Unprotected Beams

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +	Degree of shear connection:	51 %
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		

Figure 5.6 Input data of floor design zone B using the FRACOF software.

• Side A Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Edge Beam	Construction type:	Non Composite

• Side B Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 750x137 +		
Details:	h = 753 mm, b = 263 mm, t_w = 11.5 mm, t_f = 17 mm		
Beam Location:	Internal Beam	Construction type:	Composite
		Degree of shear connection:	71 %

• Side C Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Internal Beam	Construction type:	Composite
		Degree of shear connection:	51 %

• Side D Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 600 *		
Details:	h = 600 mm, b = 220 mm, t_w = 12 mm, t_f = 19 mm		
Beam Location:	Edge Beam	Construction type:	Non Composite

Note(s):
+ Minimum order: 40t per section and grade or upon agreement
* Minimum tonnage and delivery conditions upon agreement

6. Loading Details

• Normal (Cold)

Leading variable action:	5 kN/m²
Accompanying variable action:	0 kN/m²
Dead load including beam, excluding slab:	1.2 kN/m²
Calculated slab weight including mesh:	2.65 kN/m²

• Fire (Hot)

Combination factor for leading variable action:	0.5
Combination factor for other variable action:	0.3

Figure 5.7 Input data of floor design zone B using the FRACOF software.

Factored load in fire: 6.35 kN/m²

• Tabular Results

Time	Beam	Mesh	Slab top	Slab bottom	Beam capacity	Displace-ment	Slab yield	Enhance-ment	Slab capacity	Total capacity	Unity factor
mins	°C	°C	°C	°C	kN/m²	mm	kN/m²		kN/m²	kN/m²	
0	20	20	20	20	38.54	254	0.46	3.05	1.40	39.94	0.16
5	180	24	20	143	38.54	315	0.46	3.56	1.64	40.18	0.16
10	423	37	22	343	36.90	414	0.46	4.37	2.01	38.92	0.16
15	621	53	28	485	19.77	482	0.46	4.94	2.27	22.04	0.29
20	732	74	36	586	9.25	529	0.46	5.32	2.45	11.70	0.54
25	790	102	48	657	5.95	559	0.46	5.57	2.56	8.51	0.75
30	826	120	62	711	4.75	579	0.46	5.73	2.64	7.39	0.86
35	853	125	71	753	4.10	595	0.46	5.87	2.70	6.80	0.93
40	875	163	83	787	3.56	606	0.46	5.96	2.74	6.30	1.01
45	894	190	89	815	3.09	618	0.46	6.05	2.79	5.88	1.08
50	911	214	103	840	2.84	623	0.46	6.09	2.81	5.65	1.12
55	926	238	119	861	2.69	625	0.46	6.12	2.82	5.51	1.15
60	940	263	131	880	2.56	629	0.46	6.15	2.83	5.39	1.18

Maximum unity factor: 1.18 **Floor slab fails**

Figure 5.8 Results for the resistance of floor design zone B using the FRACOF software.

Figure 5.9 to Figure 5.11 shows the input and output from the FRACOF software for floor design zone B, with a ST 25C mesh size.

Considering Figure 5.11, the load bearing capacity of the slab based on the lower bound yield line mechanism has increase to 0.79 kN/m^2 due to the increased mesh area. This capacity is enhanced due to the effect of membrane action to give a slab capacity of 5.07 kN/m^2 at 60 minutes. The enhancement factor at 60 minutes was based on a slab deflection of 629 mm.

The load bearing capacity of the composite beams is added to the slab capacity to given the total load bearing capacity. The beam capacity is based on the temperature of the unprotected beams at each time step. At 60 minutes, the bending resistance of the three unprotected beams is 2.56 kN/m^2. Thus, the total load bearing resistance of the floor design zone is capacity is 5.07 + 2.56= 7.63 kN/m^2, which is greater than the applied load, hence the floor slab is adequate.

2. General Arrangement

- Spans

Span 1:	9 m
Span 2:	12 m

- Unprotected Beams

Number of internal unprotected beams:	3

3. Deck Details

- Deck Properties

Deck:	COFRAPLUS 60	Type:	Trapezoidal
Depth:	58 mm	Top flange:	106 mm
Pitch:	207 mm	Bottom flange:	62 mm
Stiffener height:	0 mm		

4. Slab Details

- Concrete

Concrete type:	Normal	Slab depth:	130 mm
		Cylinder compressive strength of concrete (f_{ck}):	25 N/mm²

- Mesh

Mesh type:	ST 25 C		
Transverse mesh area:	257 mm²/m	Bar size:	7 mm
Longitudinal mesh area:	257 mm²/m	Bar size:	7 mm
Average mesh axis distance:	30 mm	Mesh yield stress:	500 N/mm²

5. Beams Details

- Unprotected Beams

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +	Degree of shear connection:	51 %
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		

- Side A Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Edge Beam	Construction type:	Non Composite

- Side B Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 750x137 +		
Details:	h = 753 mm, b = 263 mm, t_w = 11.5 mm, t_f = 17 mm		
Beam Location:	Internal Beam	Construction type:	Composite
		Degree of shear connection:	71 %

Figure 5.9 Input data of floor design zone B using the FRACOF software.

• Side C Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Internal Beam	Construction type:	Composite
		Degree of shear connection:	51 %

• Side D Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 600 *		
Details:	h = 600 mm, b = 220 mm, t_w = 12 mm, t_f = 19 mm		
Beam Location:	Edge Beam	Construction type:	Non Composite

Note(s):
+ Minimum order: 40t per section and grade or upon agreement
* Minimum tonnage and delivery conditions upon agreement

6. Loading Details

• Normal (Cold)

Leading variable action:	5 kN/m²
Accompanying variable action:	0 kN/m²
Dead load including beam, excluding slab:	1.2 kN/m²
Calculated slab weight including mesh:	2.65 kN/m²

• Fire (Hot)

Combination factor for leading variable action:	0.5
Combination factor for other variable action:	0.3

7. Fire & Analysis

• Standard Temperature-time Curve

Fire resistance period: 60 min

Figure 5.10 Input data of floor design zone B using the FRACOF software.

Factored load in fire: 6.35 kN/m²

• Tabular Results

Time	Beam	Mesh	Slab top	Slab bottom	Beam capacity	Displace-ment	Slab yield	Enhance-ment	Slab capacity	Total capacity	Unity factor
mins	°C	°C	°C	°C	kN/m²	mm	kN/m²		kN/m²	kN/m²	
0	20	20	20	20	38.54	254	0.79	3.13	2.49	41.03	0.15
5	180	24	20	143	38.54	315	0.79	3.67	2.91	41.45	0.15
10	423	37	22	343	36.90	414	0.79	4.52	3.59	40.49	0.16
15	621	53	28	485	19.77	482	0.79	5.11	4.06	23.83	0.27
20	732	74	36	586	9.25	529	0.79	5.52	4.38	13.63	0.47
25	790	102	48	657	5.95	559	0.79	5.77	4.58	10.53	0.60
30	826	120	62	711	4.75	579	0.79	5.95	4.72	9.47	0.67
35	853	125	71	753	4.10	595	0.79	6.09	4.84	8.93	0.71
40	875	163	83	787	3.56	606	0.79	6.18	4.91	8.47	0.75
45	894	190	89	815	3.09	618	0.79	6.28	4.99	8.08	0.79
50	911	214	103	840	2.84	623	0.79	6.33	5.02	7.87	0.81
55	926	238	119	861	2.69	625	0.79	6.35	5.04	7.74	0.82
60	940	263	131	880	2.56	629	0.79	6.38	5.07	7.63	0.83

Maximum unity factor: 0.83 **Floor slab adequate**

Figure 5.11 Results for the resistance of floor design zone B using the FRACOF software.

The FRACOF software also provides a critical temperature for each of the perimeter beams, as shown in Figure 5.12. The fire protection applied to these beams should be sufficient to ensure that the temperature of the beams in a fire does not exceed this critical temperature for the required period of fire resistance. The degree of utilisation quoted for each beam is the ratio between the effect of actions on the beam in the fire condition divided by the moment resistance of the beam calculated in fire conditions at time zero (room temperature).

• Perimeter Beam Check

Side A	Section Size:	IPE 500	Non Composite Edge Beam
	Degree of Utilization:	0.58	
	Critical Temperature:	563 °C	
Side B	Section Size:	IPE 750x137	Composite Internal Beam
	Shear Connection:	71 %	
	Degree of Utilization:	0.31	
	Critical Temperature:	684 °C	
Side C	Section Size:	IPE 500	Composite Internal Beam
	Shear Connection:	51 %	
	Degree of Utilization:	0.37	
	Critical Temperature:	670 °C	
Side D	Section Size:	IPE 600	Non Composite Edge Beam
	Degree of Utilization:	0.67	
	Critical Temperature:	534 °C	

Figure 5.12 Requirements for the resistance of the perimeter beams of floor design zone B, given by the FRACOF software.

5.1.2 Floor design zone A

Figure 5.13 to Figure 5.15 shows the input and output from the FRACOF software for floor design zone B, which is 9 m by 9 m. In order to simplfy the construction ST 25C mesh reinforcement will be specified for the whole floor slab and floor design zone A is also checked for this mesh size. Within this floor design zone there are 2 unprotected composite beams.

From the output, the load bearing capacity of the slab based on the lower bound yield line mechanism is seen to be 1.03 kN/m^2. This capacity is enhanced due to the effect of membrane action to give a slab capacity of 5.39 kN/m^2 at 60 minutes. The enhancement factor at 60 minutes was based on a slab deflection of 566 mm.

The load bearing capacity of the composite beams is added to the slab capacity to given the total load bearing capacity. The beam capacity is based on the temperature of the unprotected beams at each time step. At 60 minutes, the bending resistance of the two unprotected beams is 2.56 kN/m^2. Thus, the total load bearing resistance is 2.56 + 5.39= 7.95 kN/m^2, which is greater than the applied load. The floor slab is adequate for a fire resistance of 60 minutes.

2. General Arrangement

- Spans

Span 1: 9 m

Span 2: 9 m

- Unprotected Beams

Number of internal unprotected beams: 2

3. Deck Details

- Deck Properties

Deck:	COFRAPLUS 60	Type:	Trapezoidal
Depth:	58 mm	Top flange:	106 mm
Pitch:	207 mm	Bottom flange:	62 mm
Stiffener height:	0 mm		

4. Slab Details

- Concrete

Concrete type:	Normal	Slab depth:	130 mm
		Cylinder compressive strength of concrete (f_{ck}):	25 N/mm²

- Mesh

Mesh type:	ST 25 C		
Transverse mesh area:	257 mm²/m	Bar size:	7 mm
Longitudinal mesh area:	257 mm²/m	Bar size:	7 mm
Average mesh axis distance:	30 mm	Mesh yield stress:	500 N/mm²

5. Beams Details

- Unprotected Beams

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +	Degree of shear connection:	51 %
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		

- Side A Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 550 *		
Details:	h = 550 mm, b = 210 mm, t_w = 11.1 mm, t_f = 17.2 mm		
Beam Location:	Edge Beam	Construction type:	Non Composite

- Side B Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Internal Beam	Construction type:	Composite
		Degree of shear connection:	72 %

Figure 5.13 Input data of floor design zone A using the FRACOF software.

• Side C Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Internal Beam	Construction type:	Composite
		Degree of shear connection:	51 %

• Side D Perimeter Beam

Section family:	European sections	Steel grade:	S355
Section size:	IPE 500 +		
Details:	h = 500 mm, b = 200 mm, t_w = 10.2 mm, t_f = 16 mm		
Beam Location:	Edge Beam	Construction type:	Non Composite

Note(s):
+ Minimum order: 40t per section and grade or upon agreement
* Minimum tonnage and delivery conditions upon agreement

6. Loading Details

• Normal (Cold)

Leading variable action:	5 kN/m²
Accompanying variable action:	0 kN/m²
Dead load including beam, excluding slab:	1.2 kN/m²
Calculated slab weight including mesh:	2.65 kN/m²

• Fire (Hot)

Combination factor for leading variable action:	0.5
Combination factor for other variable action:	0.3

7. Fire & Analysis

• Standard Temperature-time Curve

Fire resistance period: 60 min

Figure 5.14 Input data of floor design zone A using the FRACOF software.

Factored load in fire: 6.35 kN/m²

• Tabular Results

Time	Beam	Mesh	Slab top	Slab bottom	Beam capacity	Displace-ment	Slab yield	Enhance-ment	Slab capacity	Total capacity	Unity factor
mins	°C	°C	°C	°C	kN/m²	mm	kN/m²		kN/m²	kN/m²	
0	20	20	20	20	38.54	190	1.03	2.39	2.46	41.00	0.15
5	180	24	20	143	38.54	252	1.03	2.86	2.94	41.48	0.15
10	423	37	22	343	36.90	351	1.03	3.61	3.71	40.61	0.16
15	621	53	28	485	19.77	419	1.03	4.13	4.25	24.02	0.26
20	732	74	36	586	9.25	465	1.03	4.49	4.61	13.86	0.46
25	790	102	48	657	5.95	495	1.03	4.72	4.84	10.79	0.59
30	826	120	62	711	4.75	516	1.03	4.87	5.00	9.75	0.65
35	853	125	71	753	4.10	532	1.03	4.99	5.13	9.23	0.69
40	875	163	83	787	3.56	543	1.03	5.08	5.21	8.77	0.72
45	894	190	89	815	3.09	554	1.03	5.16	5.30	8.39	0.76
50	911	214	103	840	2.84	559	1.03	5.20	5.34	8.19	0.78
55	926	238	119	861	2.69	562	1.03	5.22	5.36	8.06	0.79
60	940	263	131	880	2.56	566	1.03	5.25	5.39	7.95	0.80

Maximum unity factor: 0.8 **Floor slab adequate**

Figure 5.15 Results for the resistance of floor design zone A using the FRACOF software.

The FRACOF software also provides a critical temperature for each of the perimeter beams, as shown in Figure 5.16. The fire protection applied to these beams should be sufficient to ensure that the temperature of the beams in a fire does not exceed this critical temperature for the required period of fire resistance. The degree of utilisation quoted for each beam is the ratio between the effect of actions on the beam in the fire condition divided by the moment resistance of the beam calculated in fire conditions at time zero (room temperature).

• Perimeter Beam Check

Side A	Section Size:	IPE 550	Non Composite Edge Beam
	Degree of Utilization:	0.38	
	Critical Temperature:	636 °C	
Side B	Section Size:	IPE 500	Composite Internal Beam
	Shear Connection:	72 %	
	Degree of Utilization:	0.37	
	Critical Temperature:	663 °C	
Side C	Section Size:	IPE 500	Composite Internal Beam
	Shear Connection:	51 %	
	Degree of Utilization:	0.31	
	Critical Temperature:	693 °C	
Side D	Section Size:	IPE 500	Non Composite Edge Beam
	Degree of Utilization:	0.62	
	Critical Temperature:	552 °C	

Figure 5.16 Requirements for the resistance of the perimeter beams of floor design zone A, given by the FRACOF software.

5.2 Reinforcement details

Since the output confirms that the load bearing capacity of zones A and B are both adequate, the ST 25C mesh provided is adequate for fire design.

This mesh has an area of 257 mm^2/m in both directions and has 7 mm wires spaced at 150 mm centres in both directions.

The mesh in this example has a yield strength of 500 N/mm^2. For fire design the Class of reinforcement should be specified as Class B or C in accordance with EN 10080, to ensure that the mesh has adequate ductility.

At joints between sheets the mesh must be adequately lapped in order to ensure that it's full tensile resistance can be developed in the event of a fire in the building. For the 7 mm diameter bars of the ST 25C mesh the minimum lap length required would be 300 mm, as shown in Table 3.1. In order to avoid the build up of bars at lapped joints, sheets of mesh with flying ends should be specified as shown in Figure 3.5.

Additional reinforcement in the form of U-shaped bars should be provided at the edge beams to ensure adequate tying between these beams and the composite slab.

5.3 Fire design of perimeter beams

5.3.1 Internal perimeter beams

The internal perimeter beams to each zone are part of more than one floor design zone. For example, if we consider the beam on Gridline B between gridlines 1 and 2 we can see from Figure 5.5 that this is member is the perimeter beam on side C of floor design zone A and is also the perimeter beam on side A of floor design zone B. The fire protection applied to this member must therefore be based on the lower value of critical temperature given by the results from these two floor design zones. Considering the FRACOF output for floor design zone B shown in Figure 5.12, the critical temperature of the beam on side A is given as 670°C. Similarly for floor design zone A, the critical temperature for the beam on side C is 693°C, as shown in Figure 5.16. In this case, floor design zone B gives the more lower and therefore more onerous critical temperature, which must be used when determining the appropriate thickness of fire protection for this member.

The following information (taken from the requirements listed in Figure 5.12) should be given to the fire protection manufacturer in order to determine the required thickness of fire protection.

Fire resistance period	60 minutes
Section size	IPE 500
Critical temperature	670°C

For this size of beam the section factor, determined in accordance with EN 1993-1-2, is:

Section Factor	104 m^{-1} box protection heated on 3 sides
	134 m^{-1} profiled protection heated on 3 sides

5.3.2 Edge beams

In this example the edge beams were designed to be non-composite. However, for the fire design case these beams should be adequately tied into the composite slab. This is achieved by providing U-bars (see Sections 3.3.2 and 3.4) and shear studs on the beam. Studs should be provided at 300mm centres where the deck is parallel to the beam and in every trough of the decking profile where the deck spans perpendicular to the beam (as recommended in Section 3.4).

The fire protection required for the edge beams should be specified in the same way as for internal perimeter beams.

5.4 Fire protection of columns

Fire protection should also be specified for all of the columns in this example. The following information should be provided when specifying the fire protection.

Fire resistance period 60 minutes

Section size HD 320 x 158

Section Factor 63 m^{-1} box protection heated on 4 sides

89 m^{-1} profiled protection heated on 4 sides

Critical temperature 500°C or 80ºC less than the critical temperature calculated on the basis of the EN 1993-1-2 design rules, whichever is the lower.

The applied fire protection should extend over the full height of the column, up to the underside of the composite floor slab.

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